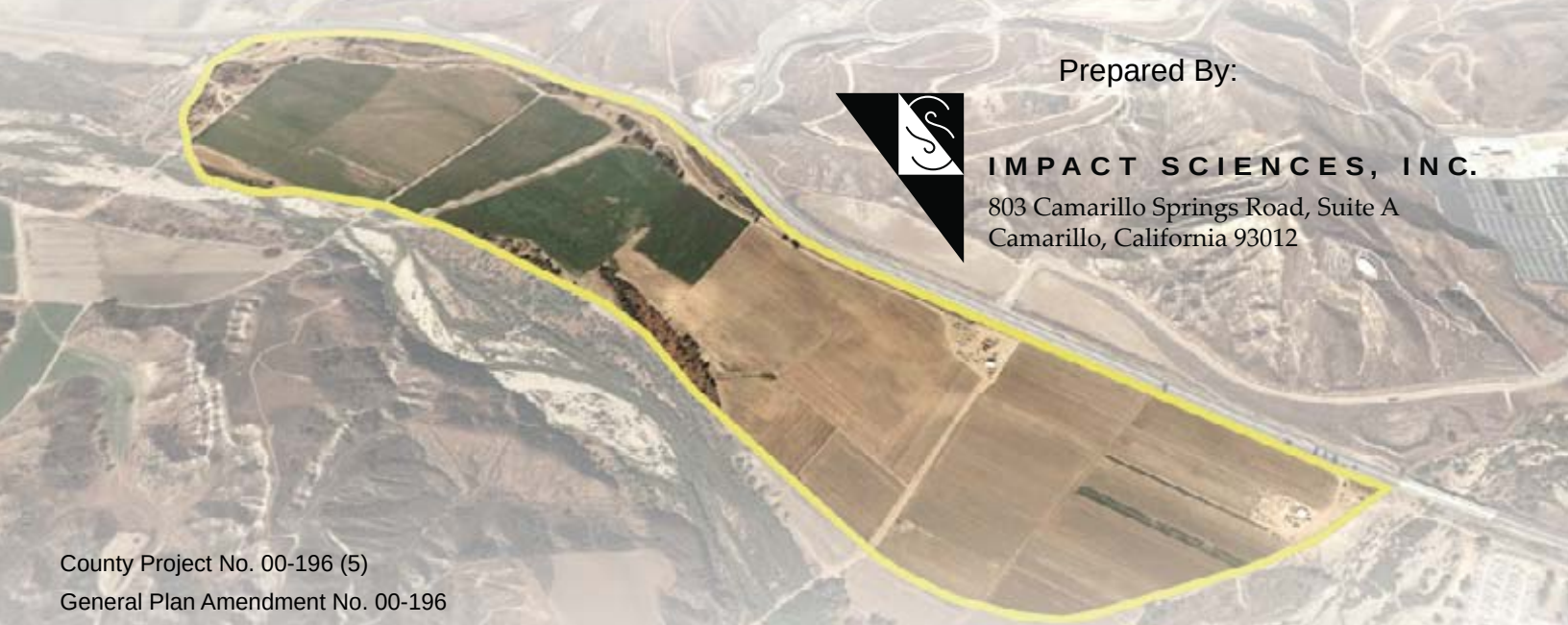
County of Los Angeles

Revised Final Environmental Impact Report

SCH No. 2004021002

Volume III
Appendices F1.0 - F4.3

LANDMARK VILLAGE



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NEWHALL RANCH
Newhall Ranch Company

September 2011

Landmark Village

Revised Final Environmental Impact Report

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Volume III

Appendices F1.0 – F4.3

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September 2011

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Project Description

CBRE Letter to Alex Herrell, January 18, 2011

COMMERCIAL REAL ESTATE SERVICES

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January 18, 2011

Mr. Alex Herrell
Newhall Land & Farming
25124 Springfield Court, Suite 300
Valencia, CA 91355

RE: Projection; Job Creation/Businesses
Entrada (VTTM 53295)
Property # 1; 53.8 acres, (PA 1-3)
Property # 2; 2.9 acres, (PA 14)

Dear Alex:

As requested I have reviewed the proposed development Site Plans for the two subject properties within **Valencia Gateway** with the intent to project the number of Jobs created and Businesses/Companies that will be encompassed within each development at "build out".

My partner (Craig Peters) and I have worked extensively in North Los Angeles/Ventura County's i.e. San Fernando, Santa Clarita and Antelope Valley's for over 20 years and have compiled a historical data base within all product designations (Office, Industrial Retail and Mixed Use) and believe the information that has been provided will be representative of the Jobs created and the Businesses/Companies that will occupy the projects at "build out".

The total GLA of both Property 1 and Property 2 is 726,000 square feet. The total jobs created encompass 2,573.

Property # 1; (GLA 676,000 square feet)

Location: North Commercial; NEC of Commerce Center Drive and Magic Mountain Pkwy
Acres: 53.8

This site is planned for mixed use Office (professional) and Commercial/Retail development.

The Office (professional) portion of the development is 400,000 square feet, will encompass four Class A steel frame buildings each 4-stories and approximating 100,000 square feet. The total GLA of the Office segment of the development will encompass 400,000 square feet.

The larger floor plates (25,000 square feet) of the 4 Class "A" buildings would most likely attract larger tenants than the average for the Santa Clarita Valley. Hence, the assumption for the average size office tenant will range between 2,500 to 10,000 square feet.

The Commercial/Retail portion of the development is 276,000 square feet and will encompass seven buildings with the buildings ranging in size from approximately 142,000 square feet to as small as 5,000 square feet.

The larger size of the proposed Commercial/Retail buildings (approximately 142,000 and 60,000 square feet) indicates a larger average size retail tenant than is typical in the area. The size range of the smaller retail tenancies will approximate 2,000 to 5,000 square feet in the multi-tenant building.

The matrix below incorporates these assumptions and projects total Jobs created and number of Businesses/Companies that will occupy the development at "build out".

Summary:

Property Type	Mixed Use Office/Retail	Jobs Created	Businesses
Number of Acres	53.8		
SF of Office:	400,000		
Average size Office tenant	7,000		
Number of Office Jobs per 1,000	4.00	1,600	
Number of Office Businesses			60
SF of Retail:	276,000		
Average size Retail tenant	14,055		
Number of Retail jobs per 1,000 SF	2.85	789	
Number of Retail Businesses			18
Total GLA, Jobs and Businesses Created	676,000	2,389	78

Property # 2; (GLA 50,000 square feet)

Location: The Old Road; across the street from Restaurant Row
Acres: 2.9

This site is also planned for mixed use Office and Commercial/Retail. The smaller size of the development and building is representative of the preponderance of the developments in the Santa Clarita Valley especially in the mid and east valley. The more typical average tenant sizes have been used. Below the matrix provides the projected Jobs created and number of Businesses/Companies that will occupy the development at "build out".

Summary:

Property Type	Mixed Use Office/Retail	Jobs Created	Businesses
Number of Acres	2.9		
SF of Office:	36,000		
Average size Office tenant	2,500		
Number of Office jobs per 1,000 SF	4.00	144	
Number of Office Businesses			14
SF of Retail:	14,000		
Average size Retail tenant	1,800		
Number of Retail jobs per 1,000 SF	2.85	40	
Number of Retail Businesses			8
Total GLA, Jobs and Businesses Created	50,000	184	22

I have also attached the aerials of the sites for your reference. Please contact me should you have any questions.

Sincerely,

CB RICHARD ELLIS, INC.

Doug Sonderegger
Executive Vice President
(818) 907-4607

CC: James E. Brown

Enclosure

APPENDIX F4.1

Geotechnical and Soil Resources

Allan E. Seward Engineering Geology, Inc.
Geologic and Geotechnical Report
December 21, 2009

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

GEOLOGIC AND GEOTECHNICAL REPORT
Review of Revised Vesting Tentative Tract Map 53108
(Dated 12/21/09)
Landmark Village
Newhall Ranch, Los Angeles County, California

VOLUME I

Prepared for:

The Newhall Land and Farming Company
23823 West Valencia Boulevard
Valencia, California 91355

Job No: 09-1702R-4
Dated December 21, 2009

27825 Smyth Drive, Valencia, California 91355 661-294-0065 FAX 661-294-0833

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ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

December 21, 2009 Job No: 09-1702R-4

The Newhall Land and Farming Company
23823 West Valencia Boulevard
Valencia, California 91355

Attention: Mr. Fred MacMurdo

Subject: **GEOLOGIC AND GEOTECHNICAL REPORT**
Review of Revised Vesting Tentative Tract Map 53108 (Dated 12/21/09)

Project: Landmark Village
Newhall Ranch
Los Angeles County, California

References: See end of text

Dear Mr. MacMurdo:

This report presents our opinions regarding the existing geologic and geotechnical conditions at the above-referenced site and their effects on the proposed development.

1.0 INTRODUCTION

The following report has been prepared at the request of The Newhall Land and Farming Company (Newhall Land) to address the revised tentative map design for Landmark Village. This firm previously prepared a Geologic and Geotechnical report dated September 27, 2000 and a subsequent addendum report dated February 10, 2001 addressing the tentative map dated June 11, 2000. This tentative map was subsequently approved from a geologic and geotechnical standpoint in review sheets dated March 1, 2001 and March 19, 2001, respectively. The revised tentative map design addressed herein was prepared by Psomas and consists of 13 sheets, dated December 21, 2009. The revised plans now include off-site grading for a borrow site and associated water tank site at Adobe Canyon to the south, a levee at Onion Field to the south, interim grading for the construction of an interchange at the intersection of Highway 126 and Chiquito Canyon Road/Long Canyon Road and associated widening of Highway 126, a utility corridor to the proposed Newhall Ranch Water Reclamation Plant (WRP) site to the west, and construction of a water tank site on PM 18108

to the northeast. All of this adjacent (off-site) property is currently owned by Newhall Land. The geologic and geotechnical conditions at the proposed borrow site, levee, and proposed improvements north of Highway 126 at Chiquito Canyon Road were previously described in our report for The Homestead project, VTT 060678 (see referenced report dated 9/30/05). The proposed Utility Corridor was addressed from a geotechnical standpoint in our referenced report (dated 5/25/07). The conclusions and recommendations presented in our previous referenced reports remain applicable, except where superceded in this report.

2.0 SCOPE OF WORK

The scope of work performed for this study included the following tasks:

1. Coordination with Newhall Land and the project supervising Civil Engineer, Psomas.

2. Review of the tentative map plans dated 12/21/09 prepared by Psomas, which utilizes updated topography prepared by Hunsaker and Associates, flown on 5/24/06 and compiled on 11/15/06.

3. Review of our previous reports for Landmark Village, the subject portions of the adjacent Homestead development (VTTM 060678), the proposed Utility Corridor, and the proposed Newhall Ranch WRP site.

4. Review of updated liquefaction analyses and incorporation of the results into our conclusions and recommendations for the site.

5. Revision of our geologic cross sections to reflect the revised, proposed grades and updated topography.

6. Updating of previous slope stability analyses at critical cross sections, based on the revised grades and topography.

7. Geologic and geotechnical review of the off-site grading proposed north of Highway 126, east of Chiquito Canyon Road, and of previous mapping and analyses for the Homestead project in this area. Preparation of pertinent cross sections, geologic and geotechnical analyses, completion of appropriate stability analyses, and development of conclusions and recommendations for the proposed grading.

8. Geologic and Geotechnical review of grading proposed for the off-site borrow area and for the associated tank site at Adobe Canyon, located south of the Santa Clara River, and of previous mapping and analyses for the Homestead project in this area. Preparation of pertinent cross sections, geologic and geotechnical analyses, completion of appropriate stability analyses, and development of conclusions and recommendations for the proposed grading.

9. Geologic and geotechnical review of the temporary levee proposed south of the Santa Clara River at the Onion Field portion of The Homestead project, based on data from our referenced report.

10. Geologic and geotechnical review of the proposed grading for the Utility Corridor design and incorporation of data and conclusions presented in our referenced report.

11. Review of the water tank site proposed at Parcel Map 18108 and a report prepared by R.T. Frankian & Associates dated 3/28/08, addressing the geologic and geotechnical conditions at the site.

12. Preparation of updated geologic/geotechnical maps for the site utilizing the revised tentative map as a base.

13. Preparation of the text of this report, which summarizes the results of our analyses and updated conclusions and recommendations for the development of the site.

14. Preparation of this report and associated figures, logs, and tables in hard copy and digital (Pdf) format.

3.0 SITE DESCRIPTION

Vesting Tentative Tract 53108, also known as Landmark Village, is a portion of the Newhall Ranch Development located in northwestern Los Angeles County. This 292.6 acre site is located south of Highway 126, north of the Santa Clara River, west of Castaic Creek and east of the Chiquito Canyon drainage. The limits of development adjacent to the Santa Clara River and Castaic Creek channels have been modified from the 6/11/00 design to provide a wider buffer between areas proposed for development and the river habitat. The site is generally flat except for existing banks between younger and older alluvium, and ascending fill slopes and local bedrock outcrops along the south side of Highway 126. The site ranges

in elevation from approximately 900 feet along the Santa Clara River on the southwestern portion of the site to a high point of 1005 ft on a knob along Highway 126 (see Tentative Map for details of the site topography). Much of the site is currently used for agricultural purposes. Portions of the northern margin of the site have been disturbed by construction associated with Highway 126, the abandoned Southern Pacific railroad line, and various pipelines. Debris, including concrete and asphalt concrete blocks, has been placed on several portions of the site. There are five abandoned oil wells and at least 13 water wells on or immediately adjacent to the site. The revised tentative map plans also include off-site grading on the Adobe Canyon, Onion Field, and Chiquito Canyon portions of the adjacent Homestead Project (VTT 060678), along the south side of Highway 126 westward to the proposed WRP site, and to the north at PM 18108 (see attached plans for details).

4.0 PROPOSED DEVELOPMENT

Vesting Tentative Tract 53108 is proposed as a residential and commercial development consisting of 422 lots that are to provide 1444 dwelling units (see Sheets 3 and 4). In addition to residential and commercial buildings, a school, a fire station, park site, open space and recreational areas, and a fire station are proposed. A lot summary of the proposed development is provided on Sheet 1 of the Tentative Map. A buried, soil-cement liner is proposed to provide bank protection along the edges of the channels of the Santa Clara River, Castaic Creek, and Chiquito Canyon drainage. A variable gradient (not steeper than 3:1) fill slope is proposed in front of the channel liner. Access to the site will be provided along Highway 126 at Wolcott Road and at Chiquito Canyon/Long Canyon Road. Final grade will be raised from 1 to 18 ft over much of the site, requiring the import of approximately 5.92 million yd^3 of fill. The tallest proposed cut and fill slopes on Landmark Village are located along the south side of Highway 126. Retaining walls up to 12 ft in height are now incorporated into the slope design, as shown on the Tentative Map. All of the proposed cut and fill slopes are less than 28 ft in height.

Additional grading has now been incorporated into the tentative map design for VTT 53108 at several off-site locations. The off-site improvements include grading to construct an interchange between Highway 126 and Chiquito Canyon Road/Long Canyon Road and associated widening of Highway 126 for an off ramp (see Sheets 3 and 11). Grading for a borrow site and associated water tank site at the Adobe Canyon portion of the Homestead project (VTT 060678) located south of the Santa Clara River, is shown on Sheet 11. A temporary levee with a soil cement liner is proposed along the south side of the Santa Clara River at the Onion Field portion of the Homestead project (See Sheet 5). A Utility Corridor

is proposed extending along the south side of Highway 126 from the west end of Landmark Village westward to the proposed WRP site (see Sheets 9 and 10). Off-site debris basins associated with the WRP site are also shown on Sheet 9. A Zone 1A water tank site is proposed on the adjacent PM 18108, as shown on Sheet 6. Geologic and geotechnical conditions at Chiquito Canyon Road, Adobe Canyon, and Onion Fields are presented in the referenced report by AESEGI for the Homestead project. The interim conditions proposed on the Landmark Village tentative map for these off-site areas are addressed herein utilizing data from our 9/30/05 Homestead report (see References). The anticipated geologic conditions for the proposed Utility Corridor are described in our referenced report (dated 5/25/07) and the proposed off-site debris basins associated with the WRP site are addressed in our referenced reports (dated 6/19/06, 9/8/06, and 1/25/08). The Zone 1A Tank site at PM 18108 has been addressed by the project geotechnical consultant for PM 18108. A copy of their report is included as **Appendix D** of this report.

5.0 GEOLOGY

The subject site is in the eastern Ventura Basin of southern California. The Ventura Basin is a westerly plunging depositional basin produced by tectonic downwarping initiated during the early Miocene and its axis approximately coincides with the Santa Clara River. All of the geologic formations exposed on the subject site were deposited within the Ventura Basin. Most of the subject site is covered by alluvium and older alluvium. Bedrock of the Pico Formation and Saugus Formation locally crops out on the site along the south side of Highway 126. The bedrock exposed at the site has been warped into an east-plunging anticinal fold. Details of the geologic units and structural at the site are provided in the referenced reports for VTT 53108 and in the referenced reports for the associated off-site areas.

6.0 SEISMIC CONSIDERATIONS

6.1 California Building Code Response Spectrum

The following parameters should be used for calculation of the California Building Code (CBC) response spectrum:

- Site Class = D [CBC, Table 1613A.5.2]
- S_s = 2.242g
- S_1 = 0.685g
- F_a = 1.0 [CBC, Table 1613.5.3 (1)]
- F_v = 1.5 [CBC, Table 1613.5.3 (2)]

The values for S_s and S_1 listed above were obtained using the Seismic Hazard Curves and Uniform Hazard Response Spectra computer program developed by the United States Geologic Survey (USGS).

6.2 Liquefaction

The potential for liquefaction at the Landmark Village site and for associated post-earthquake liquefaction-induced settlement was originally assessed for 8 of the 64 CPT soundings presented in our referenced report dated 9/27/00. We subsequently analyzed the potential for liquefaction in 21 CPT soundings utilizing current procedures set forth by the California Geological Survey and in the 2007 California Building Code. Details of our liquefaction analyses are presented in **Appendix C**.

Our analyses indicate that potential liquefaction-induced settlement will be less than one inch at 11 of the 21 CPT sounding locations that we analyzed and that potential liquefaction-induced settlement between about one and two inches could occur in the vicinity of CPT soundings 5, 13, 19, 96, 105, 106, 108, 109, 111, and 115. We have made minor revisions to the recommended alluvial removal depths shown on the attached Geotechnical Map in order to mitigate potential liquefaction-induced settlements in the vicinity of these CPT soundings.

7.0 GENERAL CONCLUSIONS AND RECOMMENDATIONS

7.1 Feasibility of Development

Vesting Tentative Tract 53108 is feasible for development from a geologic/geotechnical standpoint, provided that the recommendations presented in our prior reports and in this report are incorporated into the Tentative Map design and implemented during construction. The earthworks recommendations provided in our previous reports remain applicable, except where superceded in this report.

7.2 Earthworks Recommendations

7.2.1 Removals

Removal depth recommendations are provided on the attached Geologic/Geotechnical Map for VTT 53108 based on our previous review of the site alluvial conditions and on our updated liquefaction analyses. Ground water may be encountered during grading removals, as discussed in our previous reports. The grading contractor should be prepared to implement dewatering measures as necessary, to achieve the recommended removals. Anticipated removal depths for off-site grading are based on previous, site-specific investigations, as described herein.

7.2.2 Fill Compaction for Roadways

Roads with gradients as shallow as 0.5% are proposed at the site. As discussed in our referenced Geotechnical Letter Report (dated 8/27/07), fill depths and underlying dense alluvial deposits are expected to be relatively uniform across the site and the fill materials are anticipated to be dominantly granular in nature. Therefore, potential differential settlement is expected to have a negligible impact on the constructed gradients of the proposed roadway alignments, provided that our recommendations for removal of unsuitable soils and fill placement are followed during the grading operations. In order to further limit potential affects of fill settlement on the roadway gradients, the top 2 ft of fill materials below the road base should be compacted to at least 95% of Maximum Dry Density (per ASTM D1557).

7.2.3 Proposed Fill Slopes

Review of the revised Tentative Map indicates that no fill slopes greater than 28 ft in height are proposed on Landmark Village. All fill slopes are proposed at a gradient of 2:1 (h:v) or shallower. Based on the shear strengths of site materials and of anticipated import materials, gross stability of these fill slopes is expected to be satisfactory. However, granular alluvial soils at the site have a low cohesion and are not expected to provide satisfactory surficial stability. Therefore, the fill material within one equipment width (12 ft minimum) of the slope face should be constructed with cohesive material obtained by selective grading of on-site or imported cohesive materials.

7.2.4 Proposed Cut Slopes

Review of the revised Tentative Map design for VTT 53108 indicates that no cut slopes greater than 28 ft in height are proposed.

A 27 ft high cut slope is proposed along Highway 126 on the western portion of the site. Based on conditions observed in Bucket-Auger Boring BA-1R, a daylighted bedding component will be exposed in the Pico Formation bedrock, as shown on Cross Sections 1-1' and 2-2' (see Sheet 3). Slope stability analyses of Cross Sections 1-1' and 2-2' are presented in **Appendix B**. Results of these analyses indicate a factor of safety under static loading conditions greater than 1.5 for Cross Section 1-1'. However, owing to fill-over-cut conditions that would exist over most of this slope, a 15 ft wide stability fill with a 3 ft deep keyway is recommended. Slope stability results for Cross Section 2-2' indicate a factor of safety less than 1.5 under static loading conditions for this slope. Therefore, a 15 ft wide buttress with a 3 ft deep keyway is recommended for this slope.

Daylighted bedding conditions and older alluvium are also anticipated to be exposed in a small, 28 ft high composite cut slope located farther to the east, based on conditions observed in Boring BA-2R. Slope stability analysis of Cross Section 3-3' also indicates a factor of safety less than 1.5 under static loading conditions. In order to satisfy minimum safety factor requirements and to prevent potential surficial instability from fill-over-cut and older alluvium-over-bedrock conditions, a 15 ft wide buttress with a 3 ft deep keyway is recommended for this slope.

Minor cut slopes are proposed elsewhere on the site that will expose alluvium or artificial fill. These materials should be removed and replaced with a compacted stability fill. Therefore, no cut slopes will remain on the site following completion of grading. Selective grading and stockpiling of cohesive soils for subsequent placement in stability fills and buttress fills is recommended, as discussed in the Proposed Fill Slope section above.

7.3 Retaining Walls

Retaining walls up to 12 ft in height are now proposed along much of the proposed slope descending from Highway 126. Specific parameters for the design of these retaining walls will be provided at the grading plan stage. Gross slope stability of slopes behind and in

front of the proposed retaining walls has been evaluated. The results of these analyses are provided in **Appendix B**.

7.4 Buried Bank Stabilization

A buried soil cement liner is proposed along the southern, eastern, and western boundaries of the project in order to protect the development from erosion and damage resulting from potential storm flow events along the Santa Clara River, Castaic Creek, and Chiquito Canyon drainages respectively. The bank protection alignment and design has been modified from the Tentative Map layout addressed in our referenced 9/27/00 report. A Geotechnical report will be provided under separate cover addressing the proposed soil-cement design.

8.0 OFF-SITE GRADING

The recommendations provided below for specific off-site locations supplement the general recommendations provided in the preceding section of this report.

8.1 Onion Field Levee

A levee with buried bank stabilization is proposed along the south bank of the Santa Clara River (see Sheet 5) to protect the Onion Field area from erosion damage during storm flows. The levee is underlain by granular alluvium and older alluvium. This area was addressed from a geotechnical standpoint in our report for VTT 060678 (dated 9/30/05). Pertinent subsurface exploration locations and recommended removal depths in the vicinity of the proposed levee are shown on Sheet 5 of the attached Geologic/Geotechnical Map. Ground water may be encountered within the removal excavation for the levee. The grading contractor should be prepared to implement dewatering measures, if necessary. Removals should extend a horizontal distance equal to the removal depth behind the toe of the 5:1 (h:v) levee back slope in order to provide lateral support for the fill and to avoid the need to undercut the 5:1 fill slope during future removals for the adjacent Onion Field area. None of the recommended removals are anticipated to be deeper than the excavation for the buried liner. A detailed geotechnical report addressing the buried bank protection will be provided under separate cover.

8.2 Zone 1A Water Tank at PM 18108

A Zone 1A Water Tank is proposed at elevation 1172.0 above Franklin Parkway on PM 18108 as part of the Tentative Map for Landmark Village (see Sheet 6). A Geotechnical Report addressing the tank site was completed by the Project Geotechnical Engineer for PM 18108, R.T. Frankian and Associates dated 3/28/08. A copy of their report is attached in **Appendix D**. We have reviewed this report and accept that the findings, conclusions and recommendations presented therein are acceptable for the proposed use.

8.3 Utility Corridor to WRP Site

Sheets 9 and 10 illustrate a proposed Utility Corridor alignment extending along the south side of Highway 126 from the west end of Landmark Village to the proposed Newhall Ranch Water Reclamation Plant (WRP) site, west of San Martinez Grande Canyon. This segment of the proposed Utility Corridor was evaluated in our referenced report dated 5/25/07. Soil-cement and turf reinforcement mat are proposed along portions of the Utility Corridor to protect the utility lines from damage during storm flows along the Santa Clara River. A geotechnical evaluation of the proposed bank protection will be provided under separate cover. Proposed mass grading for the WRP Site and for the desilting basins located to the north of Highway 126 were addressed in our referenced reports for the WRP site and the associated storm drains (dated 6/19/06, 9/8/06, and 1/25/08).

8.4 Interchange at Highway 126 and Chiquito Canyon Road/Long Canyon Road

8.4.1 Introduction

Sheet 11 of the attached Tentative Map illustrates proposed grading to allow for construction of an interchange where Highway 126 crosses the ultimate alignment of Chiquito Canyon Road/Long Canyon Road. The proposed design includes widening of Highway 126 to the north to allow for construction of on ramps and off ramps. The proposed grading includes construction of south-facing cut slopes and fill slopes and small north-facing cut slopes adjacent to Highway 126, placement of disposal fill in adjacent tributary canyons and on the east side of Chiquito Canyon, and construction of necessary debris basins. Landslide Qls-XVIII(B) will be completely removed by the proposed grading.

8.4.2 Cut Slopes

Cut slopes are proposed along the north side of Highway 126 at gradients ranging from 2:1 to 3.5:1 (h:v). The large, south-facing cut slopes are designated as CS-1 through CS-4 on the attached Geotechnical Map for ease of reference. Bedding structure dips in the direction of these south-facing slopes, producing potentially adverse conditions. Four geologic cross sections (4-4' through 7-7') were therefore constructed to illustrate the subsurface bedrock geometry at critical locations for analysis. Data from two new borings (designated as B-1 and B-2) were used to help define the stratigraphy and structure of the bedrock and ground water conditions in this area. The location of the interfingering contact between the Saugus and Pico Formations was revised based on these borings.

Cut-slope CS-1: A small (30 ft high maximum), south-facing cut slope is proposed west of Chiquito Canyon Road and north of the proposed on ramp. This slope is anticipated to expose daylighted bedding conditions within the Pico Formation. Owing to the small size of the slope, a stability/buttress fill with a 15 ft wide, 3 ft deep keyway is considered adequate to provide global and surficial stability. Cut-slope CS-1 bends northward along Chiquito Canyon Road. This portion of the slope will expose neutral bedding conditions and is expected to be grossly stable.

Cut-slope CS-2: A large (175 ft high), south-facing cut slope is proposed east of Chiquito Canyon Road and north of the proposed off ramp. The lower 20 to 25 ft of this slope is proposed at a 2:1 (h:v) gradient, in general conformance with future proposed grades for VTT 060678 (in-progress). The upper portion of the slope is above future proposed grades for VTT 060678 and is designed at a gradient of 3.5:1 (h:v). Cross section 4-4' was constructed to illustrate the subsurface geologic conditions and to use for evaluating the stability of this slope. The cut slope is in close proximity to the interfingering contact between the Pico and Saugus Formations. The more conservative cross bedding strength (from the Pico Formation) and the more conservative bedding plane strength (from the Saugus Formation) were used in our stability analyses.

Analysis of cross section 4-4' indicates that cut-slope CS-2 meets minimum Los Angeles County factor of safety requirements (see **Appendix B**). However, a buttress was previously recommended for the lower portion of this slope relative to design grades for VTT 060678 and potential subsurface water conditions that were analyzed in our referenced report, dated 9/30/05. The design and need for this buttress should be

confirmed relative to the final Tentative Map design for VTT 060678. A stability fill with a 5 ft deep by 25 ft wide keyway may be needed on the lower 2:1 (h:v) portion of the slope as a precaution relative to the daylighted bedding conditions and because of potential exposure of weak interbeds along the Pico/Saugus Formation contact. Portions of Landslides Qls-XVI(B) and Qls-XVII(B) will be exposed on the eastern portion of CS-2. It is recommended that this slope be expanded to the east to allow for removal and replacement of the landslide material. Landslide Qls-XVI(B) is up slope of the proposed debris basin and should be removed to eliminate potential impacts to this basin.

Cut-slope CS-3: A 125 ft high cut slope is proposed east of CS-2. The lower two-thirds of this slope is proposed at a 2:1 (h:v) gradient in general conformance with future proposed grades for VTT 060678. The upper third of the slope is above future proposed grade for VTT 060678 and is designed at a 3.5:1 (h:v) gradient. This slope is expected to dominantly expose bedrock of the Saugus Formation. However, the Pico Formation may locally be exposed owing to the interfingering nature of the Pico/Saugus contact. The more conservative of the two formational strengths for bedding plane and cross bedding conditions were therefore used in our stability analyses. The geometry of the proposed slope relative to existing topographic and geologic conditions is illustrated on cross section 7-7', which was modified from cross section 9B-9B' from our referenced report for VTT 060678. Based on our analyses, the slope does not meet the minimum required factor of safety for stability (see **Appendix B**). A buttress with a keyway 7 ft deep and 75 ft wide will increase the factor of safety above Los Angeles County's minimum requirements. This recommended buttress design is shown on geologic cross section 7-7' (**Plate I**) and on the attached Geologic/Geotechnical Map for Sheet 11 (see **Appendix E**). A buttress was also recommended for the lower portion of this slope during our investigation for VTT 060678 (see referenced report dated 9/30/05). The design of this buttress should be confirmed relative to the final Tentative Map design for VTT 060678.

Cut-slope CS-4: A 215-ft high, composite cut and fill slope is proposed east of CS-3 on the north side of Highway 126. The cut slope areas are dissected by three narrow canyons. Placement of fill will be required in these canyons to achieve the proposed slope grades. This slope is proposed at a 2.5:1 (h:v) gradient. Cross section 5-5' and 6-6' were constructed to illustrate critical bedrock conditions relative to the proposed slope. This cut slope is expected to expose bedrock of the Saugus Formation. Analysis of cross section 5-5' indicates that the western portion of the slope does not meet the Los

Angeles County minimum factor of safety requirement for pseudo-static conditions (see **Appendix B**). A buttress with a 7 ft deep and 75 ft wide keyway will increase the factor of safety of the subject portion of CS-4 above the Los Angeles County minimum requirements. This buttress design is illustrated on geologic cross section 5-5' (see **Plate I**) and on the attached Geologic/Geotechnical Map for Sheet 11 (See **Appendix E**). Analysis of cross section 6-6' indicates that the eastern portion of the slope will have factors of safety in excess of Los Angeles County minimum requirements. However, owing to the sliver cuts and fills proposed at this slope and potentially adverse bedding conditions, a stability fill with a 55 ft wide and 5 ft deep keyway is recommended for the eastern portion of the slope.

The small, north-facing cut slopes between Highway 126 and the proposed off ramps are expected to expose antidip bedding and are anticipated to be grossly stable.

8.4.3 Disposal Fills

Disposal fill areas are proposed at four locations north of Highway 126, designated as DF-1 through DF-4 for reference (see Sheet 11 for locations). Canyon subdrains will be required for these disposal fills. The design of these drains should be addressed at the Grading Plan stage.

Disposal fill DF-1: This disposal fill is located in the small tributary canyon between cut-slopes CS-3 and CS-4 and includes construction of a fill slope connecting the two cut slopes. Much of this canyon is underlain by artificial fill associated with a pad for the Exxon Mobil Corp Castaic Junction Gas Unit #1' well. The portions of the artificial fill and any loose soil or slopewash underlying the proposed fill slope should be removed prior to placement of compacted fill. The artificial fill and any underlying soils and slopewash behind the slope should also be removed to allow placement of structural fill for future use as part of VTT 060678. Alternatively, if the artificial fill is not removed (for the intended use as a disposal fill only), the area of nonstructural fill should be designated with a Geotechnical Note to address potential future settlement. Any remaining artificial fill should be removed and recompacted prior to future development for VTT 060678. The well should be tested in the field during grading to check for leaks and abandoned in compliance with the requirements of the California Division of Oil and Gas.

Disposal fill DF-2: This disposal fill is proposed in the tributary canyon located between cut-slopes CS-2 and CS-3, and includes construction of a fill slope connecting the two cut slopes. This disposal fill overlies portions of Landslides Qls-XIII(B), Qls-XIV(B) and Qls-XVI(B). Analyses of cross sections 8-8' and 9-9' (modified from cross sections 7B-7B' and 10B-10B' for VTT 060678) indicate that these landslides do not meet the minimum Los Angeles County factor of safety requirements with placement of the proposed disposal fill. For the current intended use as a disposal fill, it is our understanding that these landslides will not be mitigated. Therefore, the landslides have been designated within a recommended Restricted Use Area on the attached Geologic/Geotechnical Map. Recommendations to mitigate these landslides relative to future proposed development for VTT 060678 are provided in our referenced report dated 9/30/05. All compressible landslide material, slopewash and alluvium must be removed prior to placement of future structural fills.

Qls-XVI(B) is a shallow landslide located on the east margin of cut-slope CS-2. Disposal fill and a debris basin are proposed to overlie this landslide. In order to mitigate potential impacts to the proposed debris basin from settlement and potential accumulation of landslide debris, it is recommended that this landslide be completely removed and the slope reconstructed as an extension of CS-2.

Disposal fill DF-3: This disposal fill is proposed to the northwest of CS-2 on the east side of Chiquito Canyon Road. A 4:1 (h:v) fill slope roughly 90 ft high is proposed with this disposal site. Based on previous analyses of higher, steeper fill slopes in this area for VTT 060678, this slope will be grossly stable per Los Angeles County criteria. This fill will overlie alluvium and Pico Formation bedrock. The alluvial removals within the footprint of this disposal fill that are recommended in our referenced report for VTT 060678 dated 9/30/05 should be completed prior to placement of compacted fill (see Sheet 11 for details).

Disposal fill DF-4: This disposal fill is proposed on the east side of Chiquito Canyon Road, roughly 1500 ft north of Highway 126. A 70-ft high 4:1 (h:v) fill slope is proposed at this site. This fill will overlie Pico Formation bedrock, landslides Qls-X(B) and Qls-XI(B), surficial failures, and alluvium. All unsuitable landslide material, artificial fill, and surficial failures within the disposal fill footprint should be removed prior to placement of compacted fill. The alluvial removals recommended in our referenced report for VTT 060678 should also be completed prior to placement of compacted fill (see Sheet 11 for details). Additional removals will likely be required

beyond the proposed disposal fill footprint at Qls-XI(B) in order to tie the fill slope into competent material.

8.5 Adobe Canyon Borrow Site and Water Tank Site

A borrow site is proposed at the Adobe Canyon portion of VTT 060678, on the south side of the Santa Clara River. This borrow site is designed to generate 6,468,500 cubic yards of raw cut and a net export of 6,322,200 cubic yards of export fill. The borrow materials will be generated almost entirely from the granular, lower member of the Saugus Formation. The fills will be generated by cutting of ridges on the flanks of Adobe Canyon, which will produce cut slopes to heights of up to 175 ft. All of these cut slopes are expected to expose antidip to neutral bedding conditions or bedding dipping steeper that the slope. The slopes will expose the lower member of the Saugus Formation, with local exposure of subhorizontal terraced deposits (see cross sections 28L-28L' through 37L-37L' on **Plate I** in **Appendix E**, which are revised from our referenced report for VTT 060678). Analysis of the highest cut slope (CS-34L) with cross section 28L-28L indicates that the antidip cut slopes will be grossly stable per Los Angeles County requirements.

Two small landslides (Qls-XIL and Qls-XIIL) are within the borrow site footprint. Most of the landslide material will be removed by proposed borrow site grading. The remainder of these landslides will be removed by the proposed grading for VTT 060678. The alluvial and artificial fill removals recommended in our referenced report for VTT 060678 for Adobe Canyon should be completed prior to future development of the site.

A water tank site is proposed at an elevation of 1170 ft near the top of cut-slope CS-36La. The cut slopes proposed adjacent to the tanks site will expose antidip to neutral bedding conditions and will be grossly stable based on analysis of the more critical conditions at cut-slope CS-34L.

9.0 LOS ANGELES COUNTY 111 STATEMENT

In compliance with Section 111 of the Los Angeles County Building Code, it is the finding of this firm that the proposed development designated on Vesting Tentative Tract 53108 will be safe against hazard from landslide, settlement or slippage and will not affect off-site property provided that all our recommendations are incorporated into the Grading Plan and implemented during construction. It is also our finding that the proposed grading shown on

off-site areas associated with the Landmark Tentative Map will be safe for the uses intended and will not affect off-site property provided that all our recommendations are incorporated in the Grading Plan and implemented during construction.

10.0 GEOLOGIST/GEOTECHNICAL ENGINEER OF RECORD

This report has been prepared assuming that Allan E. Seward Engineering Geology, Inc. will refine all geologic and geotechnical data, as required, for the Grading Plan phase of this project. If this work is performed by another party, that party must review this report, assume full responsibility for recommendations contained herein, and assume the title of responsibility as "Geologist/Geotechnical Engineer of Record" for the specific work.

11.0 LIMITATIONS

This report has been prepared by Allan E. Seward Engineering Geology, Inc. for the exclusive use of The Newhall Land and Farming Company and its design consultants for the specific site discussed herein. This report should not be considered transferable. Prior to use by others, this firm must be notified, as additional work may be required to update this report.

In the event that any modification in the location or design of the proposed development is planned, the conclusions and recommendations contained in this report will require a written review by this firm with respect to the planned modifications.

The proposed development is located in southern California, a geologically and tectonically active region, where large magnitude, potentially destructive earthquakes are common. Therefore, ground motions from moderate or large magnitude earthquakes could affect the subject site during the design life of the proposed structures.

Typically, faulting is confined to the area adjacent to a known fault. However, absolute assurance against future fault displacement in other areas is not possible in tectonically active regions because new faults can form over time and long inactive (pre-Holocene) faults may be reactivated in response to evolving tectonic stresses and geologic conditions in the earth's crust. Therefore, the location and magnitude of new ground surface ruptures during a seismic event cannot be anticipated.

In performing these professional services, this firm has used the degree of care and skill ordinarily exercised under similar circumstances by reputable geologists and geotechnical engineers practicing in this or similar localities. The data presented in this report are based on results of pertinent field and laboratory testing. It should be recognized that subsurface conditions can vary in time, and laterally, and with depth at a given site. Since the conclusions and recommendations presented in this report are based on our observations and testing, our **conclusions** and **recommendations** are **professional opinions** and are **not meant** to be a control of nature. Therefore, we make no other **warranty** either **expressed** or **implied**.

This report may not be duplicated without the written consent of this firm.

This opportunity to be of service is appreciated. Please contact us if you would like to discuss this report further.

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The following attachments and appendices complete this report.

VOLUME I

References

APPENDIX A – SUBSURFACE LOGS

Bucket-Auger Borings

- B-1 and B-2

APPENDIX B – SLOPE STABILITY

APPENDIX C – LIQUEFACTION ANALYSIS

APPENDIX D – ZONE 1A TANK SITE REPORT

- Geotechnical Grading Plan Review
  Proposed Offsite Zone 1A Water Tank Site
  Vesting Tentative Tract Map No. 53108
  Los Angeles County, California
  Dated March, 28, 2008 – JN: 99-802-21
  By R.T. Frankian & Associates

VOLUME II

APPENDIX E – CROSS SECTIONS AND MAPS

Geologic Cross Sections — Plate I

- 1-1' through 3-3': Landmark Village
- 4-4' through 9-9': Off-site Grading at Chiquito Canyon
- 28L-28L', 29L-29L', 31L-31L', 32L-32L', 33L-33L', 36L-36L' & 37L-37L': Off-site Grading @ Adobe Canyon Borrow Site

Vesting Tentative Tract Map 53108

- Cover Sheet and Notes — Sheet 1
- Typical Sections and Details — Sheet 2
- Tentative Map Plan with Geologic/Geotechnical Mapping (1"=100') — Sheets 3 and 4
- South Bank Temporary Levee with Geologic/Geotechnical Mapping — Sheet 5
- Sewer and Water Key Maps, Zone 1A Water Tank Site, and Profiles — Sheet 6
- Interim Design at Long Canyon Road and Typical Sections — Sheet 7

VOLUME III

- Wolcott Road Interchange and Chiquito Canyon Outlet Details — Sheet 8
- Off-site Improvements for Trunk Sewer/Utility Corridor to WRP Site with Geologic/Geotechnical Mapping — Sheets 9 and 10
- Off-site Grading for Chiquito and Adobe Canyons with Geologic and Geotechnical Mapping — Sheet 11
- Details and Sections — Sheet 12
- Large Lot Conveyance Map, Phasing Map & Detail Plan — Sheet 13

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1. **GEOLOGIC/GEOTECHNICAL REPORT**
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River Village, Newhall Ranch
Castaic, California
Dated September 27, 2000 – JN: 00-1702R-4

2. **GEOLOGIC AND GEOTECHNICAL REPORT – ADDENDUM NO. 1**
Response to County Comments (Review Sheets dated 12/12/00 and 1/2/01)
Vesting Tentative Tract 53108, Map Dated 6/11/00
River Village, Newhall Ranch
Castaic, California
Dated February 10, 2001 – JN: 01-1702R-4

3. **GEOLOGIC AND GEOTECHNICAL REPORT**
Review of Vesting Tentative Tract Map 060678 (Dated 8/3/05)
Vesting Tentative Tract 060678
Homestead
Newhall Ranch, California
Dated September 30, 2005 - Job No: 05-2062-4

4. **GEOLOGIC AND GEOTECHNICAL REPORT**
Review of Grading Plans (Dated 3/13/06)
Proposed Water Reclamation Plant
Newhall Ranch Project
Castaic, Los Angeles County, California
Dated June 19, 2006 - Job No: 06-1707-1

5. **GEOTECHNICAL REPORT**
Review of Storm Drain Plans
P.D. No. 2606
Proposed Water Reclamation Plant
Castaic area, Los Angeles County, California
Dated September 8, 2006 - Job No: 06-1707-1

6. **PRELIMINARY GEOLOGIC/GEOTECHNICAL REPORT**
Review of Utility Corridor Plans (30% Submittal)
Utility Corridor along Highway 126
Newhall Ranch WRP Site to Travel Village
Castaic, California
Dated May 25, 2007 – JN: 07-2079 (0)

7. **GEOTECHNICAL LETTER REPORT**
Review of Impacts of Potential Differential Settlement on Proposed Road Gradients
Landmark Village – VTTM 53108
Newhall Ranch
Los Angeles County, California
Dated August 27, 2007 – JN: 07-1702R (12)

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GEOTECHNICAL REPORT
Review of Revised Storm Drain Plans (undated) and
Rough Grading Plan (undated) for P.D. No. 2606
P.D. No. 2606
Proposed Water Reclamation Plant
Castaic Area, Los Angeles County, California
Dated January 25, 2008 – JN: 08-1707-1

Appendix A

CLIENT: The Newhall Land & Farming Company
PROJECT: Highway 126 Widening
DRILLING COMPANY: All Ways Drilling
DRILLING METHOD: Bucket Auger
HAMMER TYPE: Telescoping Kelly Bar
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2230 lb, 91-120'=1223 lb

JOB NO.: 05-2088-1 (0)
DATE: 12/21/09
LOGGED BY: MBW
DRILLED: 10/10-12/05
HOLE DIA: 24'
AVERAGE DROP: 12"
ELEVATION: 971'

DRILL HOLE LOG

BORING NO. B-1

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
						QUATERNARY ALLUVIUM; Qal (0 - 16.5')	@ 0-3' Casing
						@ 0' Yellowish-brown silty sand; loose; damp	
5		Push/ 18"		116/6.2		@ 4' Grades to pebbly sand @ 4.5' Sandy silt @ 5' Pebbly sand @ 5.5' Sandy silt with local pebbly sand channels	
10		1 1 1		122/8.1		@ 9' Stiff	
15		2 3 3		94/6.6			
				fr: N53W,33NE B: N88E,26SE		BEDROCK; TQs (16.5 - 99') @ 16.5' Light-gray, silty, fine-grained sandstone; hard; damp @ 18' Gray to olive-brown clayey mudstone; abundant caliche-lined fractures	@ 18-22' Moderately fractured
20		2 5		fr: N03E,29SE 112/16.6 B: N85E,26SE		@ 19.5' Interbedded gray to yellowish-brown, silty, fine-grained sandstone and brown mudstone	@ 22-27' Slightly fractured
				B: N87W,30SW		@ 24' Approximately 5-inch-thick gray mudstone interbed with iron-oxide staining	
25				B: N78E,26SE		@ 25' Slightly sheared mudstone interbed @ 25.5' Brown mudstone; massive	
30		5 10 15		B: N82E,24SW 126/12.9		@ 30' Slightly sheared mudstone interbed; slightly plastic	@ 30' Bulk sample Shear Test
35				Apx B: E-W,24S		@ 35' Gray silty sandstone to sandy siltstone; gradational contact with overlying and underlying units @ 36' Brown mudstone; moist; massive	LL= 51 PI= 24

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (0)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/10-12/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	BORING NO. B-1	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES; DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
				B: N83E,28SE			
40						@ 39' Grades to gray to yellowish-brown pebbly sandstone; massive; coarsens down; southward imbrication of pebbles	Shear test
		10					
45							
							@ 47-99' Surface logged due to water
50							
		6 8/5"		127/8.6			
						@ 52' Standing water	
55							
						@ 56' Brown, fine-grained sandstone; wet	
						@ 59' Silty, fine-grained sandstone	
60							
		8/5"					
65							
						@ 66' Bluish-gray, silty, fine-grained sandstone	
						@ 67' Bluish-gray siltstone	@ 67' Bulk sample
						@ 68' Brown, medium- to coarse-grained sandstone	
70							
		10 10/1"		114/16.1			
						@ 72' Brown, coarse-grained pebbly sandstone	
75							

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (0)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/10-12/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	BORING NO. B-1	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf) MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
						@ 77' Difficult to drill due to hardness @ 78' Gray, pebbly, coarse-grained sandstone	
80							
		10 8/1"					
85							
90						@ 91' Grayish-brown mudstone	@ 92' Top 2" of sample is disturbed
		20/5"		119/16.5			
95						@ 94' Gray to bluish-gray siltstone @ 97' Gray mudstone	
		29				@ 99' No recovery	
100						TOTAL DEPTH 99' (Elev. 872') Ground Water @ 52' (Elev. 919') Minor Caving from 58 to 64'	
105							
110							

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (0)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/12 & 13/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	**BORING NO.** B-2	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
						BEDROCK; TQs (0 - 116')	@ 0-5' Bulk sample
						@ 0' Yellowish-brown, sandy siltstone with pebbles; moderately hard; damp; generally massive with some faint laminations; moderately weathered	
5		3 7		Apx B: N63E, 11SE 123/3.8		@ 4' 6"-thick, brown mudstone interbed; some roots; possible old soil layer; not fully planar	
				Apx B: N56E, 17SE		@ 8' Grades sandier	
10		3 6		102/4.6		@ 10' Silty sandtone with pebbles	
15		2 3		115/2.4		@ 16' Cobbly, very friable	
20		2 2		116/4.2		@ 20' Local caliche stringers	
25						@ 24' Yellowish-brown, silty, pebbly sandstone with local cobbles; damp; hard; massive	
30		5 6		116/3.6			
35						@ 35' Grayish-brown siltstone; gradational contact with overlying and underlying units	

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (0)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/12 & 13/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	BORING NO. B-2	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
40		Bounce				@ 38' Yellowish-brown, pebbly sandstone	
45						@ 47' Cobbly; local mudstone ripups	
				Apx B: N46E,10SE		@ 48' Contact between sandstone and mudstone may be channelized	
50		17		B: N53E,15SE			
				Apx B: N49E,14SE		@ 52' Minor seepage	
55				Apx B: N59E,14SE		@ 55' 6"-thick, sandy mudstone interbed	
				B: N58E,16SE		@ 58' Yellowish-brown to gray, pebbly sandstone; massive; locally laminated	
60		25/4"		101/3.6			
65							
70		20				@ 71' Pebbly, cobbly, coarse-grained sandstone; wet	
75							

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (3)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/12 & 13/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	**BORING NO.** B-2	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
80		25		B: N58E,16SE 106/16.4		@ 80' 4"-thick, reddish-brown to gray mudstone interbed	
				B: N64E, 17SE		@ 82' 5"-thick, reddish-brown mudstone interbed	
				B: N58E,23SE		@ 83' Slightly sheared mudstone interbed; steeper than overlying laminated sandstone	
85						@ 84' Gray to reddish-brown mudstone; moist; massive	
				B: N55E,14SE			
90		25		Apx B: N60E,14SE			
95							
100							
105							@ 105-116' Surface logged due to water
110						@ 112' Greenish-gray, fine- to medium-grained sandstone; wet	

CLIENT: The Newhall Land & Farming Company	JOB NO.: 05-2088-1 (0)	**DRILL HOLE LOG**	
PROJECT: Highway 126 Widening	DATE: 12/21/09		
	LOGGED BY: MBW		
DRILLING COMPANY: All Ways Drilling	DRILLED: 10/12 & 13/05		
DRILLING METHOD: Bucket Auger	HOLE DIA: 24"		
HAMMER TYPE: Telescoping Kelly Bar	AVERAGE DROP: 12"	BORING NO. B-2	
DRIVING WEIGHTS: 0-30'=5619 lb, 31-60'=3745 lb, 61-90'=2280 lb, 91-120'=1223 lb	ELEVATION: 971'		

DEPTH (feet)	SAMPLE TYPE	BLOWS / 6"	GRAPHIC LOG	ATTITUDES, DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	USCS SYMBOL	DESCRIPTION	Remarks
115			TQs				
						TOTAL DEPTH 116' (Elev. 855') Minor Seep @ 52' Ground Water @ 71' (Elev. 900') No Caving	
120							
125							
130							
135							
140							
145							
150							

Appendix B

APPENDIX B

SLOPE STABILITY ANALYSES

Introduction

Static stability and pseudo-static stability were analyzed for cut slopes along the south side of Highway 126 and for cut slopes and natural slopes in off-site areas, including portions of Chiquito Canyon and Adobe Canyon of the adjacent Homestead Project, VTT 060678 (see attached plans in **Appendix E** for details). Cross sections that illustrate critical bedding geometry and/or slope height were selected for analysis. The Spencer method was used for calculation of the factor of safety of the analyzed slopes. A horizontal acceleration of 0.15g was used in the pseudo-static (earthquake) stability analyses. Results of the stability analyses, including slope geometry parameters, are summarized in **Table B1** – Results of Slope Stability Analyses. Slope stability diagrams that graphically illustrate the results of our analyses are attached for review.

The GSTABL7 computer program by Garry H. Gregory (Gregory Geotechnical Software) was used to calculate the factor of safety of slopes at the project site. Output is appended from the slope stability analysis runs (including a slope cross section that shows the surface with the lowest factor of safety, geotechnical parameters of soil layers, and other pertinent data). The analysis surfaces with the lowest factor of safety value are also plotted on the cross sections in **Appendix E**.

Slope Geometries and Subsurface Conditions

Stability analyses performed on the following slopes included cross-bedding and potential adverse bedding conditions, based on geologic data obtained near each cross section listed in **Table B1**. Buttresses were used in the stability analyses as mitigation for proposed cut slopes with calculated factors of safety lower than the minimum values required by Los Angeles County. The geometries of proposed cut slopes at the site include the removal of vegetation, surficial soil, Terrace Deposits, and/or bedrock materials, and placement of compacted fill on portions of slopes where buttresses or stability fills are recommended.

- **Landmark Village (VTT 53108)**

  Analyzed cut slopes within Landmark Village are shown on cross sections 1-1', 2-2', and 3-3', and are located along the south side of Highway 126. Fill-over-cut conditions are shown on cross sections 1-1' and 3-3' with retaining walls proposed at top and/or bottom of slope on each cross section. These slopes are about 20 to 28 ft in height, with

2:1 (h:v) gradients and with daylighted bedding dipping out of the slope at about 15 to 17 degrees. In order to meet Los Angeles County requirements for gross and surficial slope stability, a buttress/stability fill with a 3 ft deep by 15 ft wide keyway is recommended for each slope.

- **Adobe Canyon (VTT 060678)**

  Proposed cut-slope CS-34L, shown on cross section 28L-28L', is the highest 2:1 (h:v) cut slope in the Adobe Canyon off-site grading area (175 ft). Since favorable (antidip) bedding conditions are anticipated to be exposed in this cut slope, it meets Los Angeles County requirements for gross stability.

- **North of Highway 126 (VTT 060678)**

  Proposed cut-slopes CS-2 and CS-3 are shown on cross sections 4-4' and 7-7', respectively. Cut-slope CS-2 is a 175 ft high, combination 2:1 (h:v) and 3.5:1 (h:v) slope with potential adverse bedding dipping at about 15 degrees in the direction of the slope face. Cut-slope CS-3 is a 125 ft high, combination 2:1 (h:v) and 3.5:1 (h:v) slope with daylighted bedding dipping out of slope at about 18 degrees. In order to meet Los Angeles County requirements for gross slope stability, a buttress with a 7 ft deep by 75 ft wide keyway is recommended for cut-slope CS-3.

  Proposed cut-slope CS-4 is shown on cross sections 5-5' and 6-6'. Based on cross section 5-5', CS-4 is 215 ft in height, with a 2.5:1 (h:v) gradient and with daylighted bedding dipping out of the slope at about 17 to 21 degrees. The natural portion of the slope in the additional 20 ft above CS-4 has a 5:1 (h:v) gradient (total slope height of 236 ft). In order to meet Los Angeles County requirements for gross slope stability, a buttress with a keyway 7 ft deep by 75 ft wide is recommended for the western portion of cut-slope CS-4 per analysis of cross section 5-5'. A 5 ft deep by 55 ft wide stability fill is recommended on the eastern portion of CS-4 owing to fill-over-cut and sliver cut and fill conditions.

  Landslides Qls-XIV(B) and Qls-XIII(B) and the proposed disposal fills are shown on cross sections 8-8' and 9-9', respectively. Our analyses indicate that although the proposed fills will improve stability of these landslides, the slopes that include these landslides will not satisfy the factor of safety requirements of Los Angeles County. It is our understanding that Landslides Qls-XIV(B) and Qls-XIII(B) will not be removed or mitigated relative to the future intended use as a fill disposal site. Therefore, Restricted

APPENDIX B

Use Areas are recommended on the attached Geotechnical Maps for landslides Qls-XIV(B) and Qls-XIII(B). Recommendations for mitigation of these landslides relative to future development of the site for VTT 060678 are provided in our referenced report dated 9/30/05.

Ground Water

Review of ground water data obtained during our previous investigations and from Bucket-Auger borings B1 and B2 indicates that the proposed cut slopes in bedrock (Adobe Canyon and North of Highway 126) are above historic high ground water levels. Ground water was modeled in the slope stability analyses using a pore pressure parameter (r_u=0.12) at locations where a potential source for future ground water existed directly up gradient (updip) of the slope being analyzed.

Shear Strength and Density Parameters

Shear strength and density parameters developed for our original Review of Tentative Tract Map report for VTT 53108 (dated September 27, 2000) were used for on-site slopes located within the subject VTT 53108 and an assumed shear strength and density were adopted for select fill materials recommended for use in the stability fills and buttresses. These parameters are summarized in the following table:

Shear Strength Parameters – VTT 53108

Materials	Unit Density (pcf)	Static		Pseudo-static	
		Phi (deg)	c (psf)	Phi (deg)	c (psf)
Bedrock (Tp), parallel to bedding	130	18	200	18	200
Bedrock (Tp), cross bedding	130	31	185	45	203
Older Alluvium (Qoa)	125	28.5	151	30	200
Compacted Fill (Cef)	135	30.6	100	31	154
Select Fill	135	30	300	30	300

Shear strength and density parameters developed for our Review of Tentative Tract Map report for VTT 060678 (dated September 30, 2005) were used for slopes located within VTT 060678. These parameters are summarized in the following table:

APPENDIX B

Shear Strength Parameters – VTT 060678

Materials	Unit Weight (pcf)	Static		Pseudo-static	
		Phi (deg)	c (psf)	Phi (deg)	c (psf)
Bedrock (TQs_L), parallel to bedding	130	20	200	29	500
Bedrock (TQs_L), cross bedding	130	35	500	38	600
Bedrock (Tp), parallel to bedding	125	22	275	28	400
Bedrock (Tp), cross bedding	125	30	350	36	500
Terrace Deposits (Qt)	125	28	250	28	300
Older Alluvium (Qoa)	120	28	250	29	300
Compacted Fill (Cef)	130	30	300	30	300
Landslide Mass (Qls)	125	25	200	29	300
Slide Plane	125	12	125	15	250

The strength parameters provided above for static loading conditions are residual values and the strength values for pseudo-static loading conditions are peak values.

Surficial Stability

Permanent cut slopes that expose cut/fill transitions, daylighted bedrock, or Terrace Deposits should be constructed as stability fills. The need for additional stability fills should be evaluated during grading operations.

Conclusions

The analyzed cut slopes comply with Los Angeles County minimum requirements for gross stability under static and pseudo-static loading conditions and for surficial stability, as applicable, provided that our recommendations are followed and incorporated into project construction. The analyzed landslides do not comply with Los Angeles County minimum requirements for gross stability. The results of the stability analyses are summarized in **Table B1.**

The following attachments are located within this Appendix.

Slope Stability Analyses Results Table B1
Slope Stability Diagrams and Data Sheets for Runs 1 to 31

APPENDIX B

SLOPE STABILITY ANALYSES RESULTS

CROSS SECTION	FILE NAME	SLOPE DESIGNATION	ANALYZED SURFACE	METHOD	FACTOR OF SAFETY		COMMENTS	RUN NO.
					STATIC	PSEUDOSTATIC (COEFF = 0.15)		
1-1′	T1S1A T1PS1A	Cut slope	Bedding Plane	Spencer	1.70	1.22	Minimum safety factor for daylighted slope behind proposed retaining wall; recommended stability fill with 15′w by 3′d keyway for surficial stability.	1 2
2-2′	T2S1A T2PS1A	Cut slope	Bedding Plane	Spencer	1.52	1.06	Daylighted slope behind proposed retaining wall; Pseudo-static safety factor < 1.10; see recommended mitigation below.	3 4
2-2′	T2S2A T2PS2A	Cut slope	Circular	Spencer	1.51	1.61	Circular failure through slope less critical than bedding plane failure.	5 6
2-2′	T2S1B T2PS1B	Cut slope	Bedding Plane	Spencer	1.68	1.22	Analyses including proposed buttress / stability fill with 15′w by 3′d keyway.	7 8
3-3′	T3S1A T3PS1A	Cut slope	Bedding Plane	Spencer	1.40	1.05	Safety factor < minimum requirements; see recommended mitigation below.	9 10
3-3′	T3S2A T3PS2A	Cut slope	Circular	Spencer	1.50	1.58	Circular failure through slope less critical than bedding plane failure.	11 12
3-3′	T3S1B T3PS1B	Cut slope	Bedding Plane	Spencer	1.66	1.34	Analyses including proposed buttress / stability fill with 15′w by 3′d keyway.	13 14
4-4′	T4S1A T4PS1A	Cut slope CS-2	Bedding Plane	Spencer	1.68	1.10	Minimum safety factor.	15 16
5-5′	T5S2B T5PS2B	Cut slope CS-4	Bedding Plane	Spencer	1.56	1.07	Pseudo-static safety factor < 1.10; see recommended mitigation below.	17 18
5-5′	T5S1B T5PS1B	Cut slope CS-4	Bedding Plane	Spencer	1.66	1.14	Analyses including proposed buttress with 75′w by 7′d keyway.	19 20
6-6′	T6S1B T6PS1B	Cut slope CS-4	Bedding Plane	Spencer	1.78	1.28	Minimum safety factor.	21 22
7-7′	T7S1A	Cut slope CS-3	Bedding Plane	Spencer	1.48	--	Current design with day-lighted bedding. Minimum safety factor < 1.5. See mitigation below.	23

APPENDIX B

SLOPE STABILITY ANALYSES RESULTS

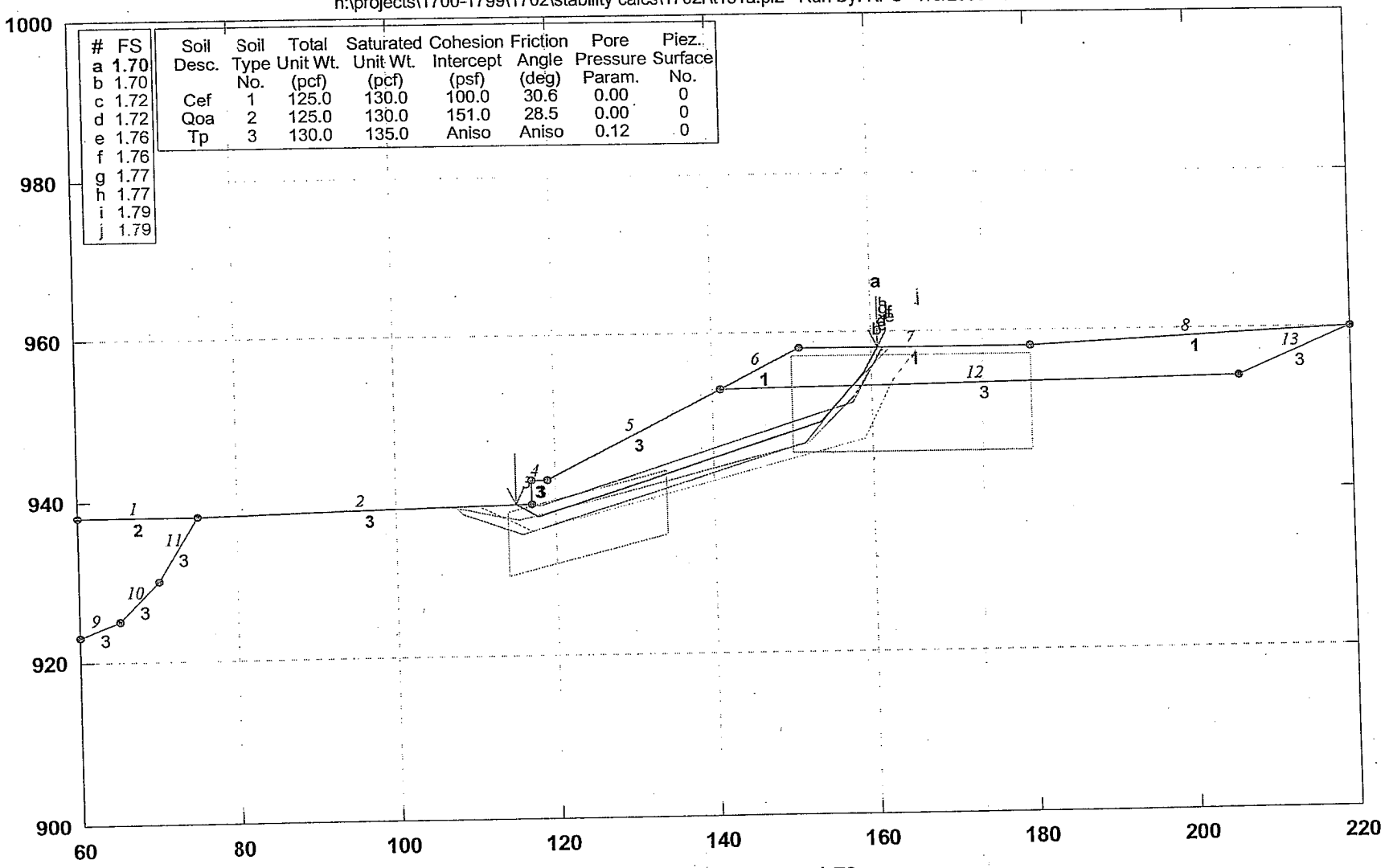
CROSS SECTION	FILE NAME	SLOPE DESIGNATION	ANALYZED SURFACE	METHOD	FACTOR OF SAFETY		COMMENTS	RUN NO.
					STATIC	PSEUDOSTATIC (COEFF = 0.15)		
7-7′	T7S2A T7PS2A	Cut slope CS-3	Bedding Plane	Spencer	1.64	1.13	Analyses including proposed buttress with 75′w by 7′d keyway.	24 25
8-8′	T8S1A	Qls-XIV(B)	Landslide Plane	Spencer	2.40		Failure surface along entire basal slide plane.	26
8-8′	T8S4B	Qls-XIV(B)	Landslide Plane	Spencer	1.43	--	Failure surface along upper portion of basal slide plane. FS < 1.5; place Qls in RUA.	27
9-9′	T9S2A	Qls-XIII(B)	Landslide Plane	Spencer	1.42	--	Failure surface along entire basal slide plane. FS < 1.5; place Qls in RUA.	28
28L-28L′	T28LS1A T28LPS1A	Cut slope CS-34L	Circular	Spencer	2.27	1.75	Highest 2:1 cut slope in Adobe Canyon area. Minimum safety factor.	29 30

TT 53108, 1-1'; static; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t1s1a.pl2 Run By: KPC 7/9/2008 03:24PM

#	FS
a	1.70
b	1.70
c	1.72
d	1.72
e	1.76
f	1.76
g	1.77
h	1.77
i	1.79
j	1.79

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	100.0	30.6	0.00	0
Qoa	2	125.0	130.0	151.0	28.5	0.00	0
Tp	3	130.0	135.0	Aniso	Aniso	0.12	0



GSTABL7 v.2 FSmin=1.70

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 1

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 7/9/2008
Time of Run: 03:24PM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t1s1a.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t1s1a.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t1s1a.PLT
PROBLEM DESCRIPTION: TT 53108, 1-1'; static; planar;
daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
8 Top Boundaries
13 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	60.00	938.00	75.00	938.00	2
2	75.00	938.00	117.00	939.00	3
3	117.00	939.00	117.01	942.00	3
4	117.01	942.00	119.00	942.00	3
5	119.00	942.00	141.00	953.00	3
6	141.00	953.00	151.00	958.00	1
7	151.00	958.00	180.00	958.00	1
8	180.00	958.00	220.00	960.00	1
9	60.00	923.00	65.00	925.00	3
10	65.00	925.00	70.00	930.00	3
11	70.00	930.00	75.00	938.00	3
12	141.00	953.00	206.00	954.00	3
13	206.00	954.00	220.00	960.00	3

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	100.0	30.6	0.00	0.0	0
2	125.0	130.0	151.0	28.5	0.00	0.0	0
3	130.0	135.0	185.0	31.0	0.12	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)
Soil Type 3 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	14.0	185.00	31.00
2	18.0	200.00	18.00
3	90.0	185.00	31.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and

C equal to zero, with water weight in the tension crack.
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 8.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	114.00	934.00	134.00	939.00	8.00
2	150.00	951.00	180.00	951.00	12.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 34
Number of Trial Surfaces with Misleading FS = 2
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 3.6 %
Statistical Data On All Valid FS Values:
FS Max = 7.342 FS Min = 1.697 FS Ave = 2.871
Standard Deviation = 0.651 Coefficient of Variation = 22.66 %
((Simplified Janbu FS for Critical Surface = 1.572))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	115.09	938.95
2	117.89	937.37
3	153.59	948.76
4	158.94	954.71
5	160.88	958.00

*** FOS = 1.697 Theta (ki=1.0) = 19.00 ***
Lambda = 0.344
Individual data on the 10 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.9	139.4	0.0	19.2	0.	0.	0.0	0.0	0.0
2	0.0	3.4	0.0	0.5	0.	0.	0.0	0.0	0.0
3	0.9	501.0	0.0	69.0	0.	0.	0.0	0.0	0.0
4	1.1	641.7	0.0	80.8	0.	0.	0.0	0.0	0.0
5	22.0	17913.9	0.0	2256.3	0.	0.	0.0	0.0	0.0
6	10.0	11788.1	0.0	1484.8	0.	0.	0.0	0.0	0.0
7	2.6	3192.9	0.0	402.2	0.	0.	0.0	0.0	0.0
8	4.0	3576.1	0.0	641.9	0.	0.	0.0	0.0	0.0
9	1.3	656.5	0.0	0.0	0.	0.	0.0	0.0	0.0
10	1.9	398.0	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	115.09	938.95
2	117.89	937.37
3	153.59	948.76
4	158.94	954.71

```
     5        160.88      958.00
***  FOS =      1.697   Theta (ki=1.0) =     19.00  ***
                    Lambda =    0.344
```

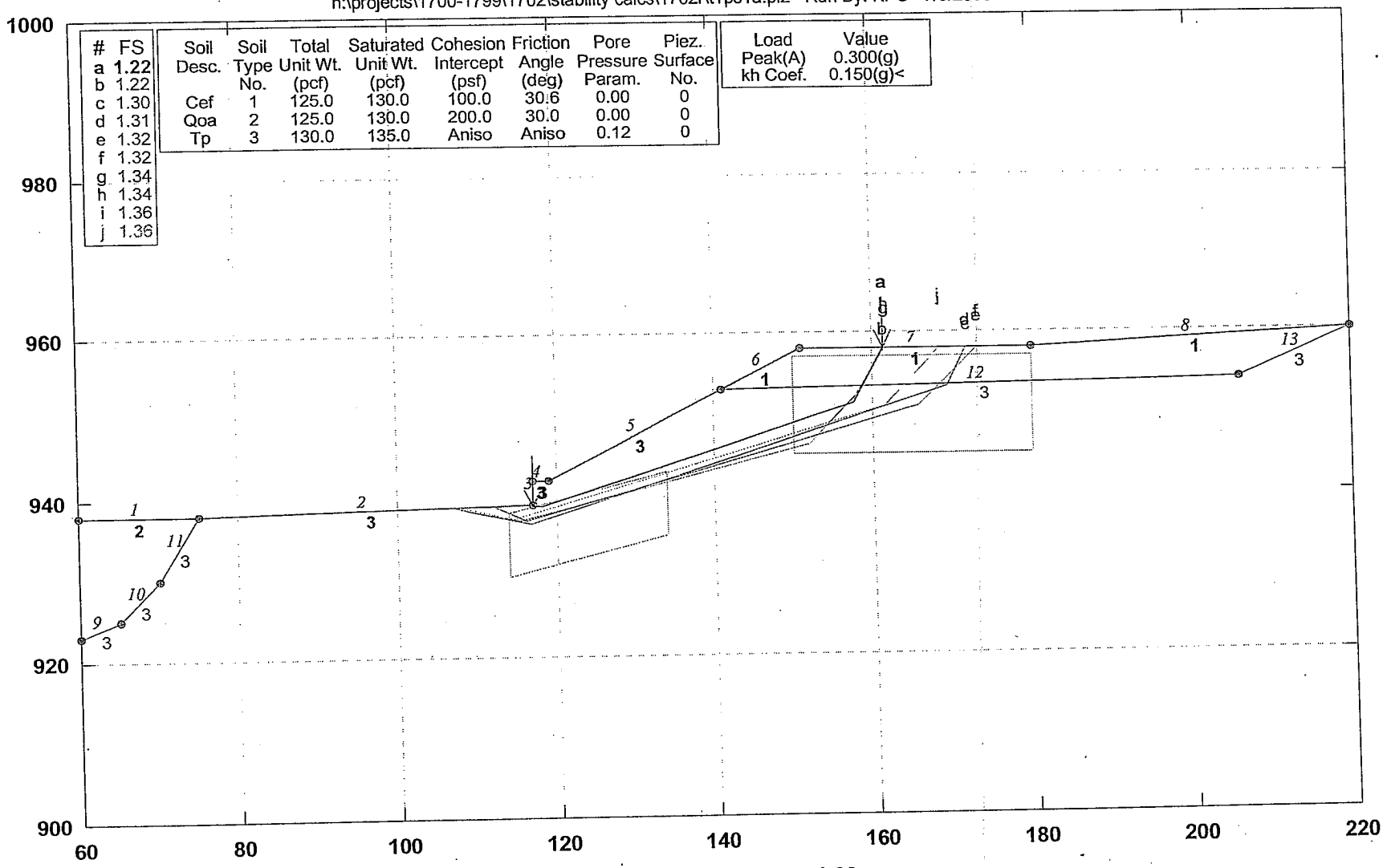
TT 53108, 1-1'; pseudo; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t1ps1a.pl2 Run By: KPC 7/9/2008 03:25PM

#	FS
a	1.22
b	1.22
c	1.30
d	1.31
e	1.32
f	1.32
g	1.34
h	1.34
i	1.36
j	1.36

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	100.0	30.6	0.00	0
Qoa	2	125.0	130.0	200.0	30.0	0.00	0
Tp	3	130.0	135.0	Aniso	Aniso	0.12	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.22

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 2

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                03:25PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t1ps1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t1ps1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t1ps1a.PLT
PROBLEM DESCRIPTION:  TT 53108, 1-1'; pseudo; planar;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    8 Top   Boundaries
   13 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
    1         60.00      938.00       75.00      938.00         2
    2         75.00      938.00      117.00      939.00         3
    3        117.00      939.00      117.01      942.00         3
    4        117.01      942.00      119.00      942.00         3
    5        119.00      942.00      141.00      953.00         3
    6        141.00      953.00      151.00      958.00         -
    7        151.00      958.00      180.00      958.00         1
    8        180.00      958.00      220.00      960.00         1
    9         60.00      923.00       65.00      925.00         3
   10         65.00      925.00       70.00      930.00         3
   11         70.00      930.00       75.00      938.00         3
   12        141.00      953.00      206.00      954.00         3
   13        206.00      954.00      220.00      960.00         3
 User Specified Y-Origin =        900.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  3 Type(s) of Soil
 Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle   Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.    (psf)     No.
   1   125.0    130.0     100.0     30.6     0.00      0.0       0
   2   125.0    130.0     200.0     30.0     0.00      0.0       0
   3   130.0    135.0     203.0     45.0     0.12      0.0       0
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  3 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction     Counterclockwise     Cohesion     Friction
   Range       Direction Limit     Intercept      Angle
    No.            (deg)             (psf)        (deg)
     1              14.0             203.00        45.00
     2              18.0             200.00        18.00
     3              90.0             203.00        45.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
```

C equal to zero, with water weight in the tension crack.
Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 8.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	114.00	934.00	134.00	939.00	8.00
2	150.00	951.00	180.00	951.00	12.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 145
Number of Trial Surfaces with Misleading FS = 10
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 15.5 %
Statistical Data On All Valid FS Values:
FS Max = 6.485 FS Min = 1.215 FS Ave = 2.891
Standard Deviation = 0.667 Coefficient of Variation = 23.08 %
((Simplified Janbu FS for Critical Surface = 1.183))
Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	117.09	938.89
2	118.10	938.71
3	157.60	951.13
4	161.35	958.00

*** FOS = 1.215 Theta (ki=1.0) = 18.53 ***
Lambda = 0.335
Individual data on the 8 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	2.1	0.0	0.3	0.	0.	0.3	0.0	0.0
2	1.1	454.3	0.0	55.2	0.	0.	68.1	0.0	0.0
3	0.9	368.7	0.0	46.4	0.	0.	55.3	0.0	0.0
4	22.0	14450.1	0.0	1817.7	0.	0.	2167.5	0.0	0.0
5	10.0	10307.5	0.0	1296.6	0.	0.	1546.1	0.0	0.0
6	6.6	6631.3	0.0	834.2	0.	0.	994.7	0.0	0.0
7	1.2	855.5	0.0	214.5	0.	0.	128.3	0.0	0.0
8	2.6	761.3	0.0	0.0	0.	0.	114.2	0.0	0.0

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	117.00	938.89
2	118.10	938.71
3	157.60	951.13

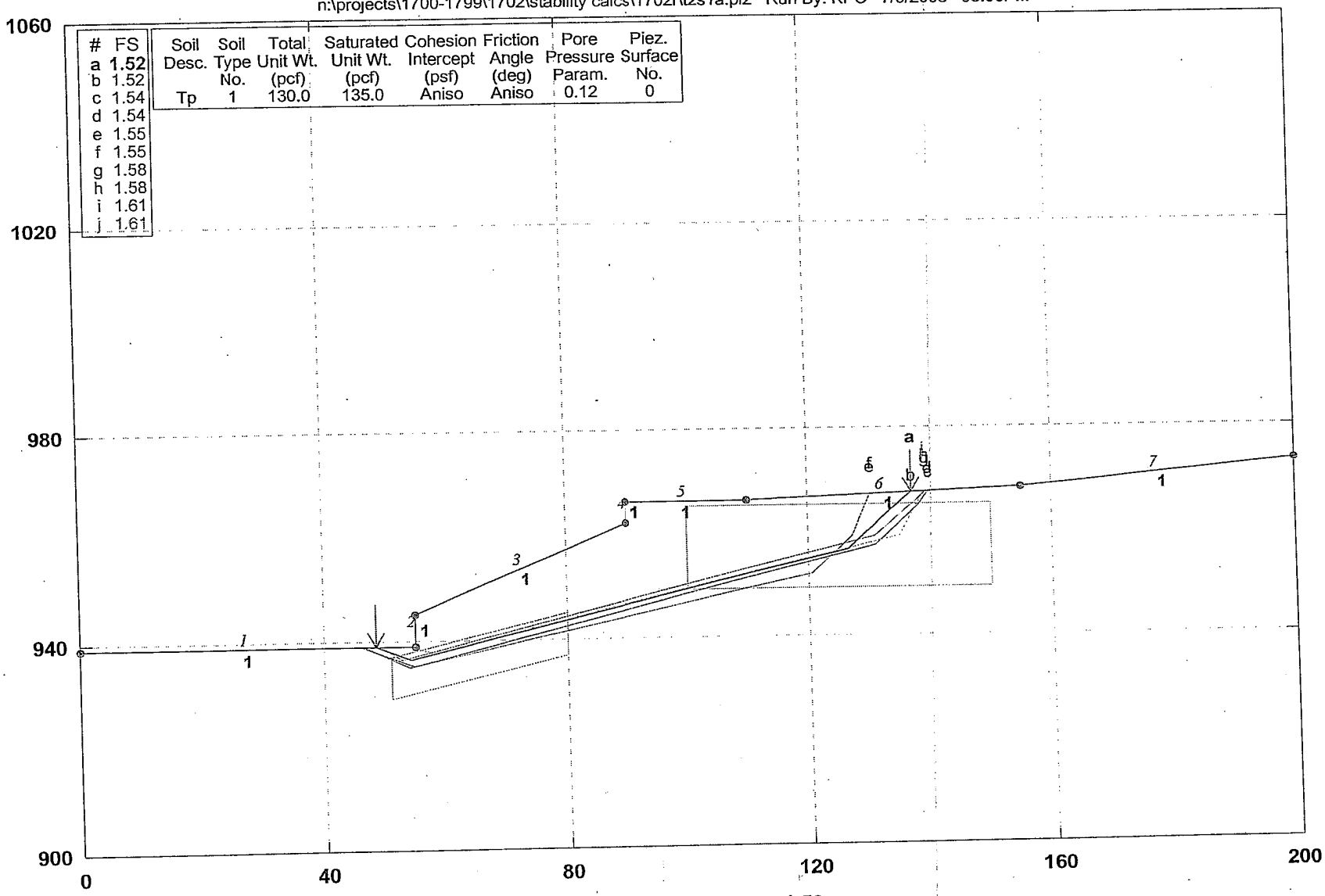
```
     4          161.35       958.00
***  FOS =        1.215    Theta (ki=1.0) =     18.53  ***
                        Lambda =    0.335
```

TT 53108, 2-2'; static; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t2s1a.pl2 Run By: KPC 7/9/2008 03:06PM

#	FS
a	1.52
b	1.52
c	1.54
d	1.54
e	1.55
f	1.55
g	1.58
h	1.58
i	1.61
j	1.61

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0



GSTABL7 v.2 FSmin=1.52

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 3

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                03:06PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2s1a.PLT
PROBLEM DESCRIPTION:  TT 53108, 2-2'; static; planar;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    7 Top   Boundaries
    7 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
    1          0.00      939.00       55.00      939.00         1
    2         55.00      939.00       55.01      945.00         1
    3         55.01      945.00       90.00      962.00         1
    4         90.00      962.00       90.01      966.00         1
    5         90.01      966.00      110.00      966.00         1
    6        110.00      966.00      155.00      968.00         1
    7        155.00      968.00      200.00      973.00         1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.   (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
  1   130.0    135.0     185.0     31.0    0.12       0.0       0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
 Soil Type  1 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction     Counterclockwise     Cohesion      Friction
   Range       Direction Limit     Intercept       Angle
    No.            (deg)             (psf)         (deg)
     1              13.5             185.00         31.00
     2              16.5             200.00         18.00
     3              90.0             185.00         31.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Sliding Block Surfaces, Has Been
 Specified.
 1000 Trial Surfaces Have Been Generated.
 2 Boxes Specified For Generation Of Central Block Base
 Length Of Line Segments For Active And Passive Portions Of
 Sliding Block Is  10.0
```

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	51.00	933.00	80.00	941.00	8.00
2	100.00	957.00	150.00	957.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 21
Number of Trial Surfaces with Misleading FS = 2
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 2.3 %
Statistical Data On All Valid FS Values:
FS Max = 6.622 FS Min = 1.516 FS Ave = 2.738
Standard Deviation = 0.758 Coefficient of Variation = 27.70 %
((Simplified Janbu FS for Critical Surface = 1.428))

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	48.52	939.00
2	54.23	936.41
3	126.45	956.62
4	133.52	963.70
5	137.02	967.20

*** FOS = 1.516 Theta (ki=1.0) = 15.36 ***
Lambda = 0.275

Individual data on the 9 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.7	961.3	0.0	126.7	0.	0.	0.0	0.0	0.0
2	0.8	247.2	0.0	30.8	0.	0.	0.0	0.0	0.0
3	0.0	7.0	0.0	0.9	0.	0.	0.0	0.0	0.0
4	35.0	54470.0	0.0	6787.6	0.	0.	0.0	0.0	0.0
5	0.0	22.9	0.0	2.8	0.	0.	0.0	0.0	0.0
6	20.0	43600.2	0.0	5433.1	0.	0.	0.0	0.0	0.0
7	16.4	25754.5	0.0	3209.3	0.	0.	0.0	0.0	0.0
8	7.1	6184.6	0.0	1049.6	0.	0.	0.0	0.0	0.0
9	3.5	762.6	0.0	129.5	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	48.52	939.00
2	54.23	936.41
3	126.45	956.62
4	133.52	963.70
5	137.02	967.20

*** FOS = 1.516 Theta (ki=1.0) = 15.36 ***
Lambda = 0.275

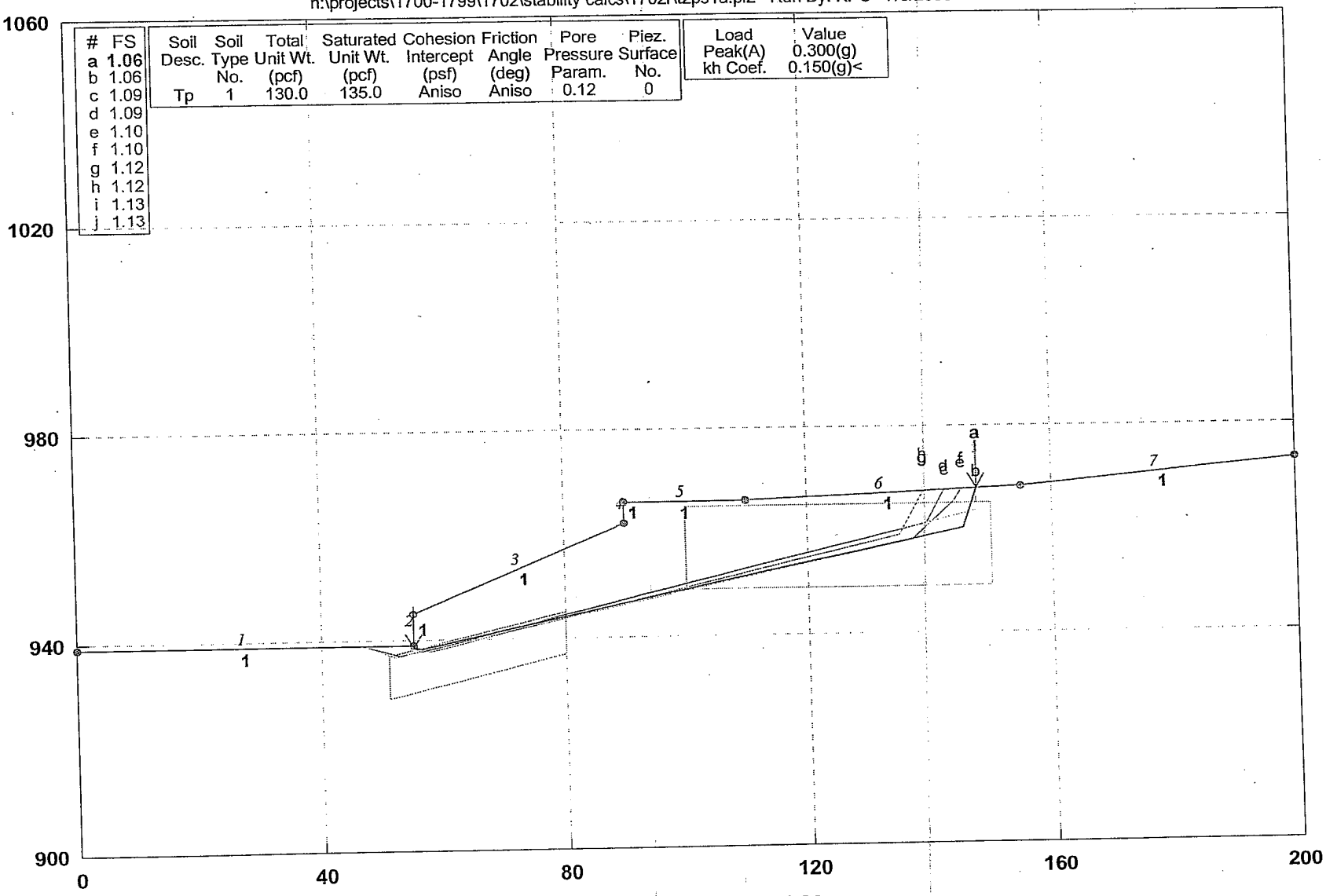
TT 53108, 2-2'; pseudo; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t2ps1a.pl2 Run By: KPC 7/9/2008 03:07PM

#	FS
a	1.06
b	1.06
c	1.09
d	1.09
e	1.10
f	1.10
g	1.12
h	1.12
i	1.13
j	1.13

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.06

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 4

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 7/9/2008
Time of Run: 03:07PM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2psla.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2psla.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2psla.PLT
PROBLEM DESCRIPTION: TR 53108, 2-2'; pseudo; planar;
daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
7 Top Boundaries
7 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	939.00	55.00	939.00	1
2	55.00	939.00	55.01	945.00	1
3	55.01	945.00	90.00	962.00	1
4	90.00	962.00	90.01	966.00	1
5	90.01	966.00	110.00	966.00	1
6	110.00	966.00	155.00	968.00	1
7	155.00	968.00	200.00	973.00	1

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	130.0	135.0	203.0	45.0	0.12	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)
Soil Type 1 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	13.5	203.00	45.00
2	16.5	200.00	18.00
3	90.0	203.00	45.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.
Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	51.00	933.00	80.00	941.00	8.00
2	100.00	957.00	150.00	957.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 163
Number of Trial Surfaces with Misleading FS = 7
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 17.0 %
Statistical Data On All Valid FS Values:
FS Max = 5.922 FS Min = 1.064 FS Ave = 2.738
Standard Deviation = 0.685 Coefficient of Variation = 25.03 %
((Simplified Janbu FS for Critical Surface = 1.018))
Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	938.53
2	56.27	936.29
3	145.59	960.25
4	147.77	967.68

*** FOS = 1.064 Theta (ki=1.0) = 13.00 ***
Lambda = 0.231
Individual data on the 7 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	4.5	0.0	0.6	0.	0.	0.7	0.0	0.0
2	1.3	1132.9	0.0	138.4	0.	0.	169.9	0.0	0.0
3	33.7	49869.1	0.0	6162.6	0.	0.	7480.4	0.0	0.0
4	0.0	22.7	0.0	2.8	0.	0.	3.4	0.0	0.0
5	20.0	44074.6	0.0	5446.5	0.	0.	6611.2	0.0	0.0
6	35.6	50513.1	0.0	6242.1	0.	0.	7577.0	0.0	0.0
7	2.2	1037.1	0.0	442.8	0.	0.	155.6	0.0	0.0

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	938.53
2	56.27	938.29
3	145.59	960.25
4	147.77	967.68

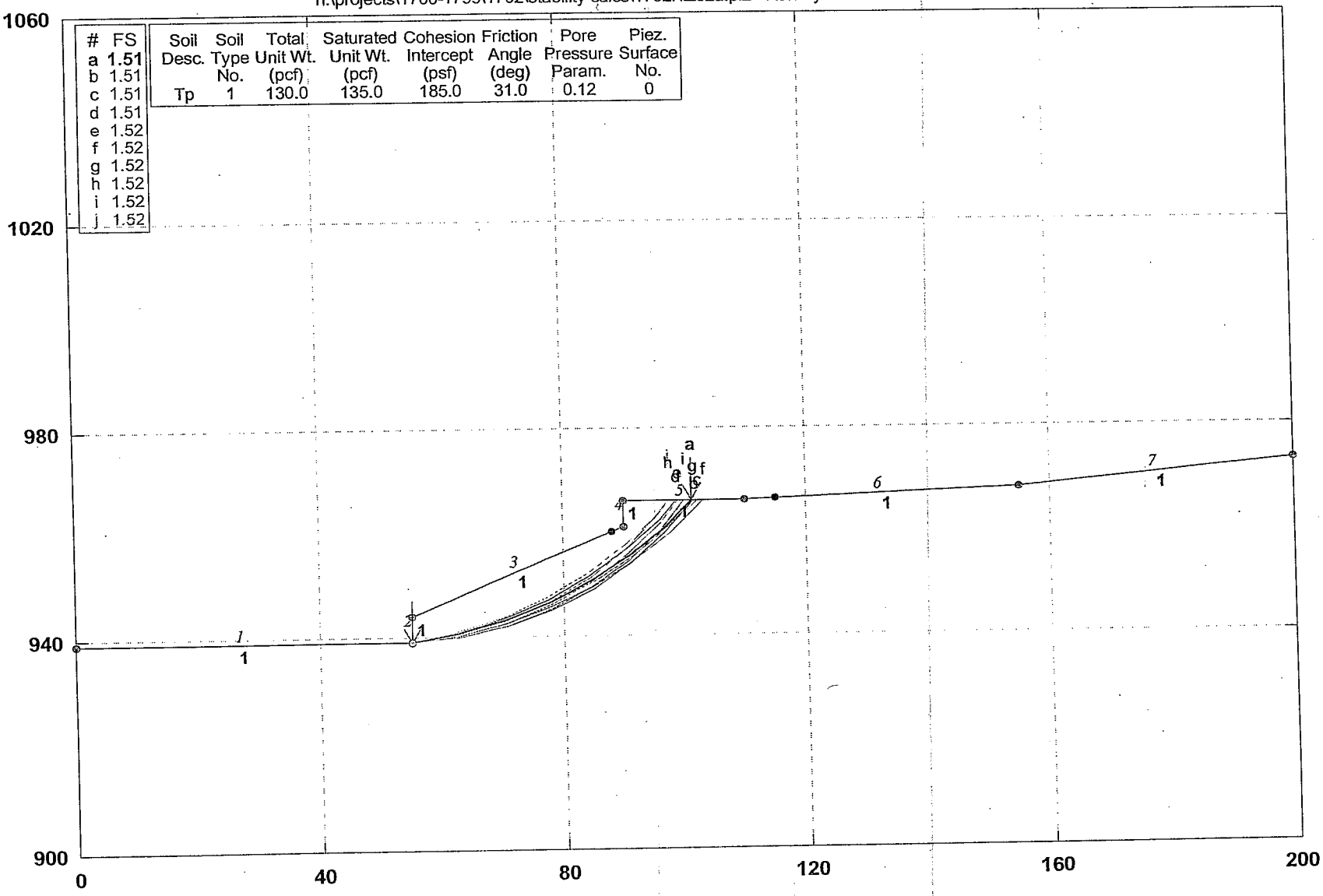
*** FOS = 1.064 Theta (ki=1.0) = 13.00 ***
Lambda = 0.231

TT 53108, 2-2'; static; circular; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t2s2a.pl2 Run By: KPC 7/8/2008 09:24AM

#	FS
a	1.51
b	1.51
c	1.51
d	1.51
e	1.52
f	1.52
g	1.52
h	1.52
i	1.52
j	1.52

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	185.0	31.0	0.12	0



GSTABL7 v.2 FSmin=1.51

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 5

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
  ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/8/2008
Time of Run:                09:24AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2s2a.PLT
PROBLEM DESCRIPTION:  TT 53108, 2-2'; static; circular;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    7 Top   Boundaries
    7 Total Boundaries
Boundary      X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
    1          0.00      939.00       55.00      939.00         1
    2         55.00      939.00       55.01      944.00         1
    3         55.01      944.00       90.00      961.00         1
    4         90.00      961.00       90.01      966.00         1
    5         90.01      966.00      110.00      966.00         1
    6        110.00      966.00      155.00      968.00         1
    7        155.00      968.00      200.00      973.00         1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
  1   130.0    135.0     185.0    31.0    0.12       0.0       0
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 500 Trial Surfaces Have Been Generated.
 500 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
Along The Ground Surface Between  X =  55.00(ft)
                              and X =  55.00(ft)
Each Surface Terminates Between   X =  80.00(ft)
                              and X = 115.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =       0.00(ft)
 8.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -25.0
And  15.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
      Selected Ki function = Bi-linear
      Selected Lambda Coefficient =  1.00
      The option of no water in the tension crack on
      the last slice has been selected.
      Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
```

(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 1
Number of Trial Surfaces with Misleading FS = 1
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.4 %
Statistical Data On All Valid FS Values:
FS Max = 2.363 FS Min = 1.507 FS Ave = 1.962
Standard Deviation = 0.260 Coefficient of Variation = 13.26 %
((Modified Bishop FS for Critical Surface = 1.508))
Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	939.00
2	62.82	940.70
3	70.41	943.22
4	77.69	946.54
5	84.57	950.62
6	90.97	955.42
7	96.82	960.87
8	101.26	966.00

Circle Center At X = 43.25 ; Y = 1012.25 ; and Radius = 74.18
*** FOS = 1.507 Theta (ki=1.0) = 27.07 ***
Lambda = 0.511
Individual data on the 10 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	3.2	0.0	0.4	0.	0.	0.0	0.0	0.0
2	7.8	6139.5	0.0	753.9	0.	0.	0.0	0.0	0.0
3	7.6	7581.0	0.0	958.6	0.	0.	0.0	0.0	0.0
4	7.3	7920.0	0.0	1044.7	0.	0.	0.0	0.0	0.0
5	6.9	7251.8	0.0	1011.9	0.	0.	0.0	0.0	0.0
6	5.4	4959.8	0.0	743.6	0.	0.	0.0	0.0	0.0
7	0.0	11.5	0.0	1.7	0.	0.	0.0	0.0	0.0
8	1.0	1366.1	0.0	204.8	0.	0.	0.0	0.0	0.0
9	5.9	5974.9	0.0	980.2	0.	0.	0.0	0.0	0.0
10	4.4	1477.7	0.0	271.0	0.	0.	0.0	0.0	0.0

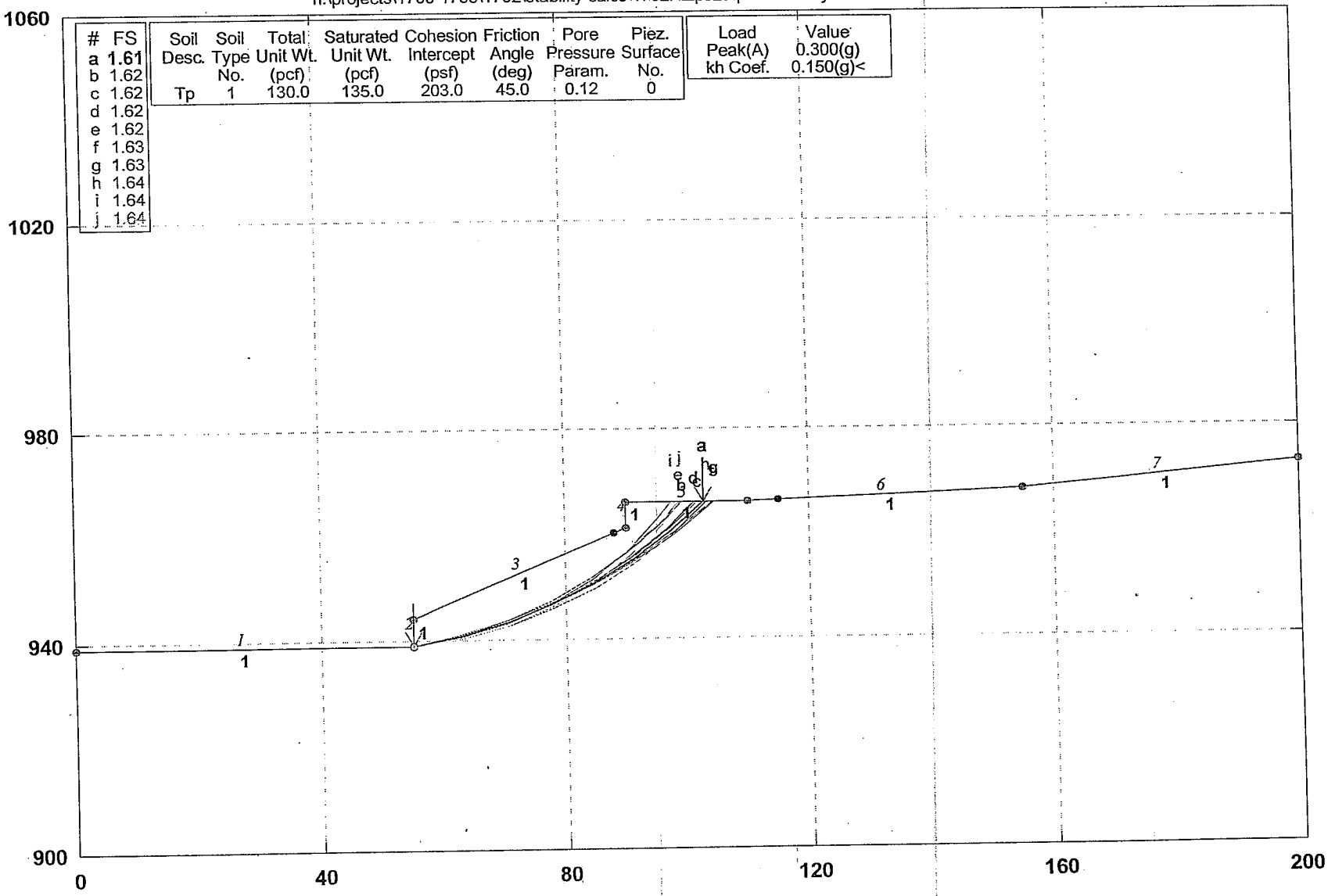
TT 53108, 2-2'; cut slope; pseudo; circular; Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t2ps2a.pl2 Run By: KPC 7/8/2008 09:41AM

#	FS
a	1.61
b	1.62
c	1.62
d	1.62
e	1.62
f	1.63
g	1.63
h	1.64
i	1.64
j	1.64

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	203.0	45.0	0.12	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.61

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

RUN No. 6

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/8/2008
Time of Run:                09:41AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2ps2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2ps2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2ps2a.PLT
PROBLEM DESCRIPTION:  TT 53108, 2-2'; cut slope; pseudo;
                      circular; Ru=0.12

BOUNDARY COORDINATES
    7 Top   Boundaries
    7 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
    1         0.00      939.00       55.00      939.00         1
    2        55.00      939.00       55.01      944.00         1
    3        55.01      944.00       90.00      961.00         1
    4        90.00      961.00       90.01      966.00         1
    5        90.01      966.00      110.00      966.00         1
    6       110.00      966.00      155.00      968.00         1
    7       155.00      968.00      200.00      973.00         1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
 1 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)    Param.    (psf)     No.
  1   130.0    135.0     203.0     45.0     0.12       0.0      0
Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
Specified Seismic Pore-Pressure Factor =   0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 500 Trial Surfaces Have Been Generated.
 500 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
Along The Ground Surface Between  X =  55.00(ft)
                             and  X =  55.00(ft)
Each Surface Terminates Between   X =  88.00(ft)
                             and  X = 115.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =       0.00(ft)
 8.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -25.0
And  15.0 deg.
Factor Of Safety Calculation By GLE (Spencer's) Method of Slices
Has Gone Through 20 Iterations
 Theta       FOS         FOS
 (deg)     (Moment)    (Force)
(ki=1.0)   (Equil.)    (Equil.)   Lambda
 13.00       1.772       1.592     0.231
```

19.50	1.730	1.605	0.354
39.02	2.159	1.658	0.810
10.90	1.781	1.588	0.193
0.00	1.809	1.569	0.000
73.73	1.812	1.809	3.426
75.02	1.354	0.692	3.737
73.72	1.294	1.295	3.424
73.29	-0.769	1.198	3.332
73.86	1.319	1.331	3.457
73.87	1.098	1.726	3.458
13.17	1.771	1.593	0.234
10.84	1.781	1.588	0.192
52.56	2.334	1.725	1.306
0.00	1.809	1.569	0.000
0.00	1.809	1.569	0.000
0.00	1.809	1.564	0.000
0.00	1.809	1.564	0.000
0.00	1.809	1.564	0.000
0.00	1.809	1.564	0.000

The Trial Failure Surface In Question Is Defined
By The Following 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	939.00
2	62.83	940.66
3	70.44	943.13
4	77.75	946.38
5	84.68	950.37
6	91.15	955.07
7	97.10	960.41
8	102.14	966.00

Factor Of Safety Calculation By GLE (Spencer's) Method of Slices
Has Gone Through 20 Iterations

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
13.00	1.742	1.594	0.231
19.50	1.714	1.606	0.354
43.37	2.078	1.668	0.945
8.10	1.755	1.586	0.142
0.00	1.769	1.573	0.000
77.37	1.637	1.655	4.464
68.81	2.214	1.885	2.580
76.79	1.745	1.811	4.259
75.01	2.289	2.069	3.735
76.24	1.264	1.263	4.083
76.25	0.042	1.048	4.087
17.61	1.724	1.603	0.317
6.47	1.758	1.583	0.113
54.64	2.393	1.723	1.409
0.00	1.769	1.573	0.000
0.00	1.769	1.573	0.000
40.63	1.635	1.659	0.858
34.74	1.473	1.641	0.693
43.90	1.809	1.670	0.962
38.37	1.125	1.651	0.792

The Trial Failure Surface In Question Is Defined
By The Following 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	939.00
2	62.76	940.95
3	70.24	943.78
4	77.34	947.46
5	83.97	951.94
6	90.03	957.17
7	95.45	963.05

8 97.58 966.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 2
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.4 %
Statistical Data On All Valid FS Values:
FS Max = 2.463 FS Min = 1.608 FS Ave = 2.094
Standard Deviation = 0.258 Coefficient of Variation = 12.31 %
((Modified Bishop FS for Critical Surface = 1.639))
Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	939.00
2	62.81	940.73
3	70.41	943.22
4	77.73	946.46
5	84.69	950.41
6	91.22	955.02
7	97.26	960.27
8	102.68	966.00

Circle Center At X = 41.54 ; Y = 1018.37 ; and Radius = 80.50
*** FOS = 1.608 Theta (ki=1.0) = 19.47 ***
Lambda = 0.353
Individual data on the 10 slices

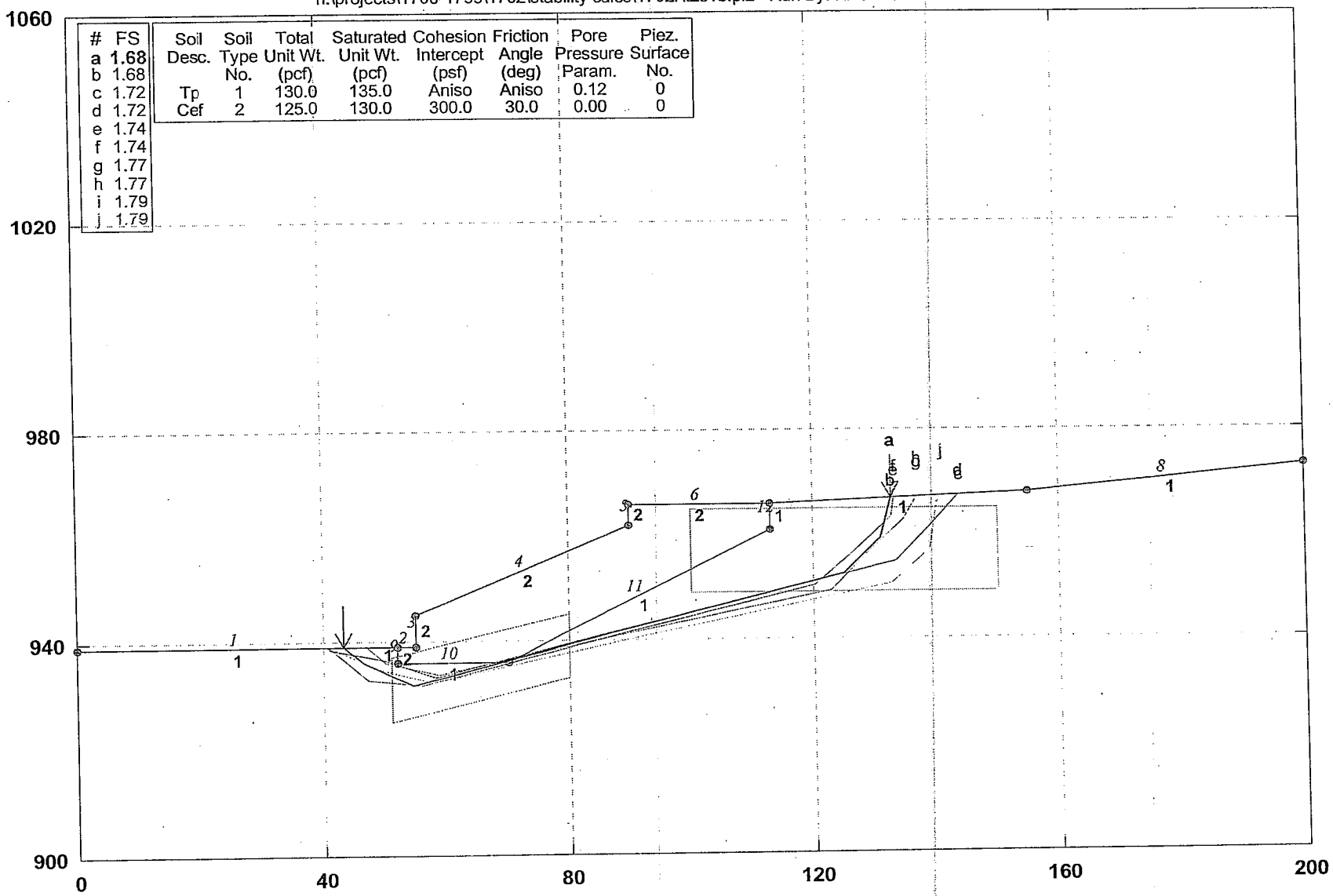
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan. (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	3.2	0.0	0.4	0.	0.	0.5	0.0	0.0
2	7.8	6115.0	0.0	751.5	0.	0.	917.2	0.0	0.0
3	7.6	7569.9	0.0	956.1	0.	0.	1135.5	0.0	0.0
4	7.3	8007.0	0.0	1050.7	0.	0.	1201.0	0.0	0.0
5	7.0	7504.0	0.0	1035.2	0.	0.	1125.6	0.0	0.0
6	5.3	5128.6	0.0	753.6	0.	0.	769.3	0.0	0.0
7	0.0	12.1	0.0	1.8	0.	0.	1.8	0.0	0.0
8	1.2	1793.8	0.0	263.6	0.	0.	269.1	0.0	0.0
9	6.0	6563.2	0.0	1042.7	0.	0.	984.5	0.0	0.0
10	5.4	2017.9	0.0	352.7	0.	0.	302.7	0.0	0.0

TT 53108, 2-2'; static; planar; Ru=0.12; stab. fill w/ 3'd by 15'w key

n:\projects\1700-1799\1702\stability calcs\1702r\t2s1b.pl2 Run By: KPC 7/10/2008 11:29AM

#	FS
a	1.68
b	1.68
c	1.72
d	1.72
e	1.74
f	1.74
g	1.77
h	1.77
i	1.79
j	1.79

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Cef	2	125.0	130.0	300.0	30.0	0.00	0



GSTABL7 v.2 FSmin=1.68

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 7

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                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
  ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                11:29AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2s1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2s1b.PLT
PROBLEM DESCRIPTION:  TT 53108, 2-2'; static; planar;
                      Ru=0.12; stab. fill w/ 3'd by 15'w key

BOUNDARY COORDINATES
    8 Top   Boundaries
   12 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)     Below Bnd
    1         0.00      939.00      52.00       939.00        2
    2        52.00      939.00      55.00       939.00        2
    3        55.00      939.00      55.01       945.00        2
    4        55.01      945.00      90.00       962.00        2
    5        90.00      962.00      90.01       966.00        2
    6        90.01      966.00     113.00       966.00        2
    7       113.00      966.00     155.00       968.00        1
    8       155.00      968.00     200.00       973.00        1
    9        52.00      939.00      52.01       936.00        1
   10        52.01      936.00      70.00       936.00        1
   11        70.00      936.00     112.99       961.00        1
   12       112.99      961.00     113.00       966.00        1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
  1   130.0    135.0     185.0     31.0    0.12       0.0       0
  2   125.0    130.0     300.0     30.0    0.00       0.0       0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
Soil Type  1 Is Anisotropic
Number Of Direction Ranges Specified =  3
Direction     Counterclockwise      Cohesion      Friction
  Range       Direction Limit     Intercept       Angle
   No.            (deg)              (psf)         (deg)
    1             13.5              185.00         31.00
    2             16.5              200.00         18.00
    3             90.0              185.00         31.00
ANISOTROPIC SOIL NOTES:
   (1) An input value of 0.01 for C and/or Phi will cause Aniso
       C and/or Phi to be ignored in that range.
   (2) An input value of 0.02 for Phi will set both Phi and
       C equal to zero, with no water weight in the tension crack.
   (3) An input value of 0.03 for Phi will set both Phi and
       C equal to zero, with water weight in the tension crack.
A Critical Failure Surface Searching Method, Using A Random
```

Technique For Generating Sliding Block Surfaces, Has Been
Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 9.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	51.00	931.00	80.00	939.00	12.00
2	100.00	957.00	150.00	957.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 44
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 4.4 %
Statistical Data On All Valid FS Values:
FS Max = 8.709 FS Min = 1.682 FS Ave = 3.037
Standard Deviation = 0.904 Coefficient of Variation = 29.78 %
((Simplified Janbu FS for Critical Surface = 1.539))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	43.24	939.00
2	46.56	935.93
3	54.59	931.86
4	124.91	952.48
5	130.95	959.15
6	132.69	966.94

*** FOS = 1.682 Theta (ki=1.0) = 13.02 ***
Lambda = 0.231
Individual data on the 15 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	3.3	662.3	0.0	108.3	0.	0.	0.0	0.0	0.0
2	5.4	3145.9	0.0	423.3	0.	0.	0.0	0.0	0.0
3	0.0	7.5	0.0	1.0	0.	0.	0.0	0.0	0.0
4	2.6	2135.0	0.0	287.3	0.	0.	0.0	0.0	0.0
5	0.4	373.2	0.0	46.7	0.	0.	0.0	0.0	0.0
6	0.0	12.7	0.0	1.6	0.	0.	0.0	0.0	0.0
7	13.7	24687.9	0.0	3087.2	0.	0.	0.0	0.0	0.0
8	2.6	5172.9	0.0	0.0	0.	0.	0.0	0.0	0.0
9	18.7	42203.1	0.0	5277.5	0.	0.	0.0	0.0	0.0
10	0.0	27.5	0.0	3.4	0.	0.	0.0	0.0	0.0
11	23.0	59563.4	0.0	7448.4	0.	0.	0.0	0.0	0.0
12	0.0	22.0	0.0	2.8	0.	0.	0.0	0.0	0.0
13	11.9	24085.2	0.0	3011.9	0.	0.	0.0	0.0	0.0
14	6.0	8559.9	0.0	1530.3	0.	0.	0.0	0.0	0.0
15	1.7	869.6	0.0	479.9	0.	0.	0.0	0.0	0.0

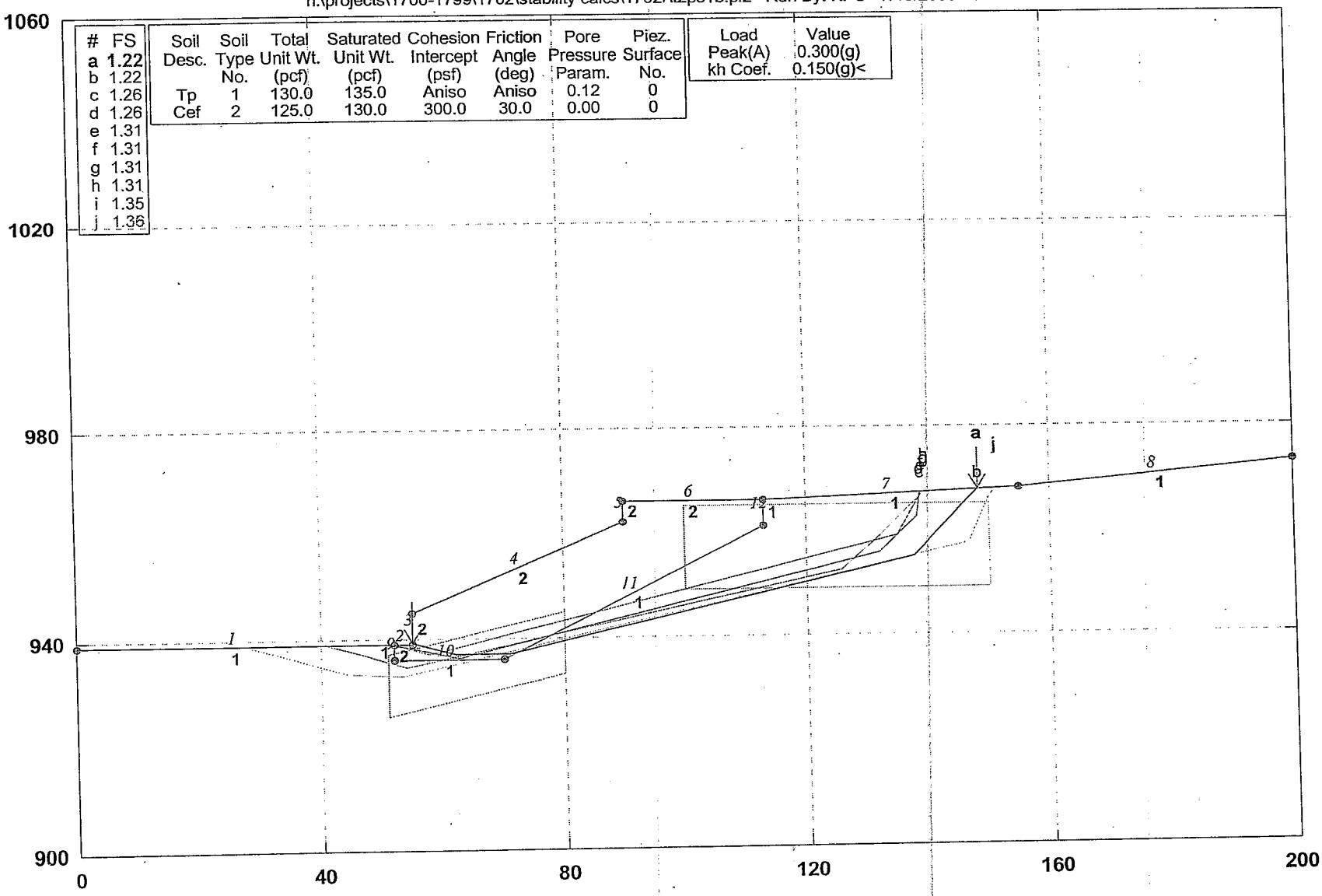
TT 53108, 2-2'; pseudo; planar; Ru=0.12; stab. fill w/ 3'd by 15'w key

n:\projects\1700-1799\1702\stability calcs\1702r\t2ps1b.pl2 Run By: KPC 7/10/2008 11:31AM

#	FS
a	1.22
b	1.22
c	1.26
d	1.26
e	1.31
f	1.31
g	1.31
h	1.31
i	1.35
j	1.36

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Cef	2	125.0	130.0	300.0	30.0	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.22

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 8

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
             (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:        7/10/2008
Time of Run:              11:31AM
Run By:                   KPC
Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2ps1b.in
Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\t2ps1b.OUT
Unit System:              English
Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t2ps1b.PLT
PROBLEM DESCRIPTION:  TT 53108, 2-2'; pseudo; planar;
                      Ru=0.12; stab. fill w/ 3'd by 15'w key

BOUNDARY COORDINATES
    8 Top   Boundaries
   12 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1         0.00      939.00      52.00       939.00        1
    2        52.00      939.00      55.00       939.00        2
    3        55.00      939.00      55.01       945.00        2
    4        55.01      945.00      90.00       962.00        2
    5        90.00      962.00      90.01       966.00        2
    6        90.01      966.00     113.00       966.00        2
    7       113.00      966.00     155.00       968.00        1
    8       155.00      968.00     200.00       973.00        1
    9        52.00      939.00      52.01       936.00        1
   10        52.01      936.00      70.00       936.00        1
   11        70.00      936.00     112.99       961.00        1
   12       112.99      961.00     113.00       966.00        1
 User Specified Y-Origin =        900.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1   130.0    135.0     203.0     45.0    0.12       0.0      0
   2   125.0    130.0     300.0     30.0    0.00       0.0      0
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  1 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction    Counterclockwise      Cohesion      Friction
   Range      Direction Limit     Intercept       Angle
    No.           (deg)             (psf)         (deg)
     1            13.5             203.00         45.00
     2            16.5             200.00         18.00
     3            90.0             203.00         45.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
```

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 9.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	51.00	931.00	80.00	939.00	12.00
2	100.00	937.00	150.00	957.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 157
Number of Trial Surfaces with Misleading FS = 13
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 17.0 %
Statistical Data On All Valid FS Values:
FS Max = 5.607 FS Min = 1.221 FS Ave = 2.639
Standard Deviation = 0.710 Coefficient of Variation = 26.89 %
((Simplified Janbu FS for Critical Surface = 1.170))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	55.00	939.35
2	61.97	937.11
3	70.97	936.95
4	137.73	955.05
5	143.42	962.02
6	148.18	967.68

*** FOS = 1.221 Theta (ki=1.0) = 11.76 ***
Lambda = 0.208
Individual data on the 11 slices

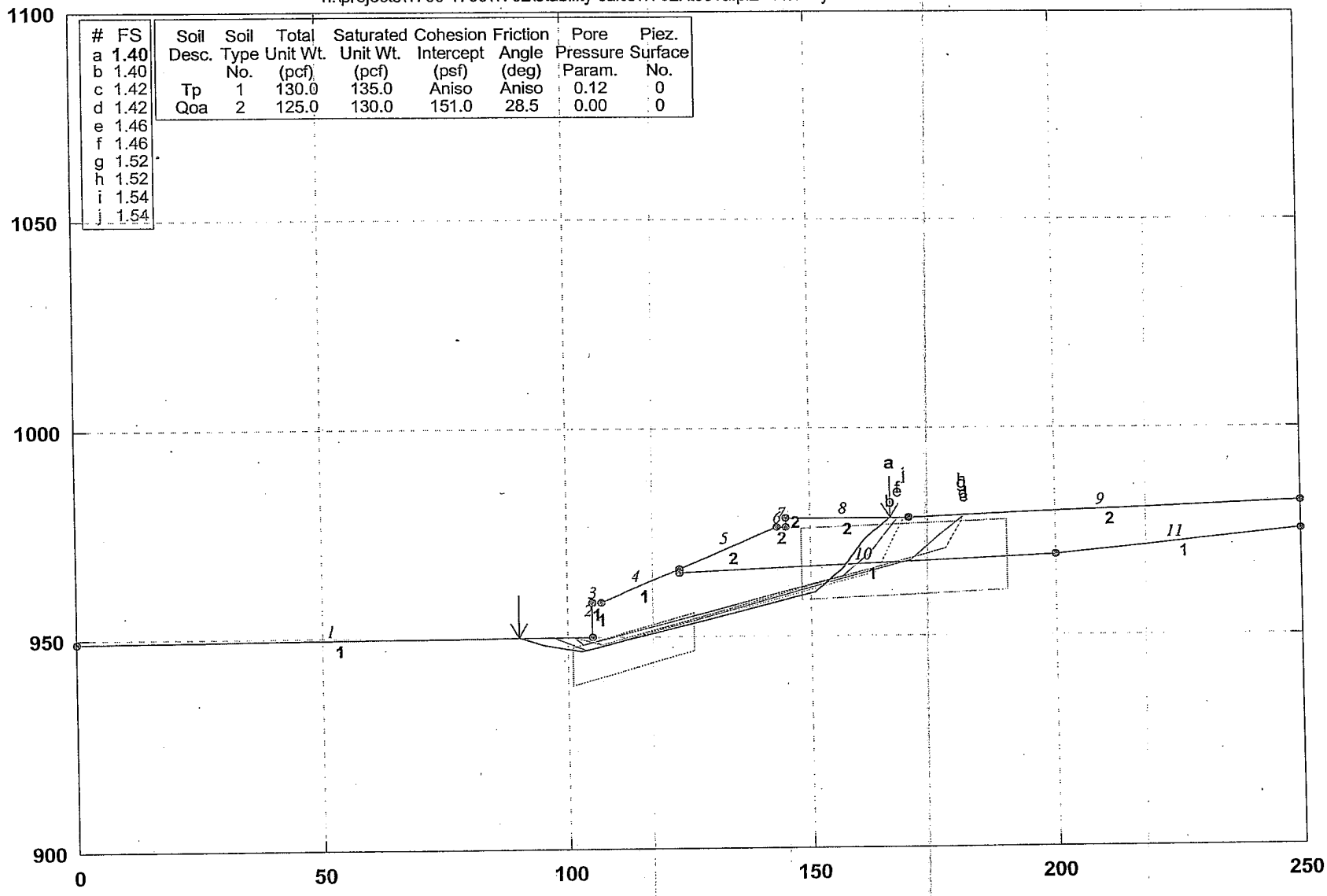
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	3.3	0.0	0.0	0.	0.	0.5	0.0	0.0
2	7.0	7367.3	0.0	0.0	0.	0.	1105.1	0.0	0.0
3	9.0	15229.7	0.0	0.0	0.	0.	2284.5	0.0	0.0
4	1.2	2459.8	0.0	0.0	0.	0.	369.0	0.0	0.0
5	17.8	40244.2	0.0	5003.6	0.	0.	6036.6	0.0	0.0
6	0.0	27.6	0.0	3.4	0.	0.	4.1	0.0	0.0
7	23.0	60723.1	0.0	7549.8	0.	0.	9108.5	0.0	0.0
8	0.0	22.8	0.0	2.8	0.	0.	3.4	0.0	0.0
9	24.7	47883.4	0.0	5953.4	0.	0.	7182.5	0.0	0.0
10	5.7	6492.1	0.0	1232.7	0.	0.	973.8	0.0	0.0
11	4.8	1679.9	0.0	313.0	0.	0.	252.0	0.0	0.0

TT 53108, 3-3'; static; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t3s1a.pl2 Run By: KPC 7/9/2008 02:51PM

#	FS
a	1.40
b	1.40
c	1.42
d	1.42
e	1.46
f	1.46
g	1.52
h	1.52
i	1.54
j	1.54

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Qoa	2	125.0	130.0	151.0	28.5	0.00	0



GSTABL7 v.2 FSmin=1.40

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 9

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 7/9/2008
Time of Run: 02:51PM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s1a.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s1a.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t3s1a.PLT
PROBLEM DESCRIPTION: TT 53108, 3-3'; static; planar;
daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
9 Top Boundaries
11 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	949.00	105.00	950.00	1
2	105.00	950.00	105.01	958.00	1
3	105.01	958.00	107.00	958.00	1
4	107.00	958.00	123.00	966.00	1
5	123.00	966.00	143.00	976.00	2
6	143.00	976.00	145.00	976.00	2
7	145.00	976.00	145.01	978.00	2
8	145.01	978.00	170.00	978.00	2
9	170.00	978.00	250.00	982.00	2
10	123.00	965.00	200.00	969.00	1
11	200.00	969.00	250.00	975.00	1

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	130.0	135.0	185.0	31.0	0.12	0.0	0
2	125.0	130.0	151.0	28.5	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)
Soil Type 1 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	15.5	185.00	31.00
2	18.5	200.00	18.00
3	90.0	185.00	31.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been

Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 8.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	101.00	943.00	126.00	951.00	9.00
2	148.00	967.00	190.00	969.00	17.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 62
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 6.2 %
Statistical Data On All Valid FS Values:
FS Max = 6.363 FS Min = 1.401 FS Ave = 2.752
Standard Deviation = 0.744 Coefficient of Variation = 27.04 %
((Simplified Janbu FS for Critical Surface = 1.313))
Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	89.85	949.86
2	94.93	948.06
3	102.79	946.60
4	150.91	960.37
5	156.54	966.06
6	160.92	972.75
7	166.16	978.00

*** FOS = 1.401 Theta (ki=1.0) = 16.72 ***
Lambda = 0.300
Individual data on the 14 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.1	607.7	0.0	77.3	0.	0.	0.0	0.0	0.0
2	7.9	2667.3	0.0	325.5	0.	0.	0.0	0.0	0.0
3	2.2	881.0	0.0	110.0	0.	0.	0.0	0.0	0.0
4	0.0	8.8	0.0	1.1	0.	0.	0.0	0.0	0.0
5	2.0	2710.6	0.0	338.3	0.	0.	0.0	0.0	0.0
6	16.0	24761.7	0.0	3090.6	0.	0.	0.0	0.0	0.0
7	20.0	40415.6	0.0	5044.4	0.	0.	0.0	0.0	0.0
8	2.0	4479.1	0.0	559.1	0.	0.	0.0	0.0	0.0
9	0.0	23.3	0.0	2.9	0.	0.	0.0	0.0	0.0
10	5.9	13824.5	0.0	1725.5	0.	0.	0.0	0.0	0.0
11	5.6	10493.7	0.0	1790.8	0.	0.	0.0	0.0	0.0
12	0.5	675.2	0.0	147.8	0.	0.	0.0	0.0	0.0
13	3.9	4040.6	0.0	0.0	0.	0.	0.0	0.0	0.0
14	5.2	1721.0	0.0	0.0	0.	0.	0.0	0.0	0.0

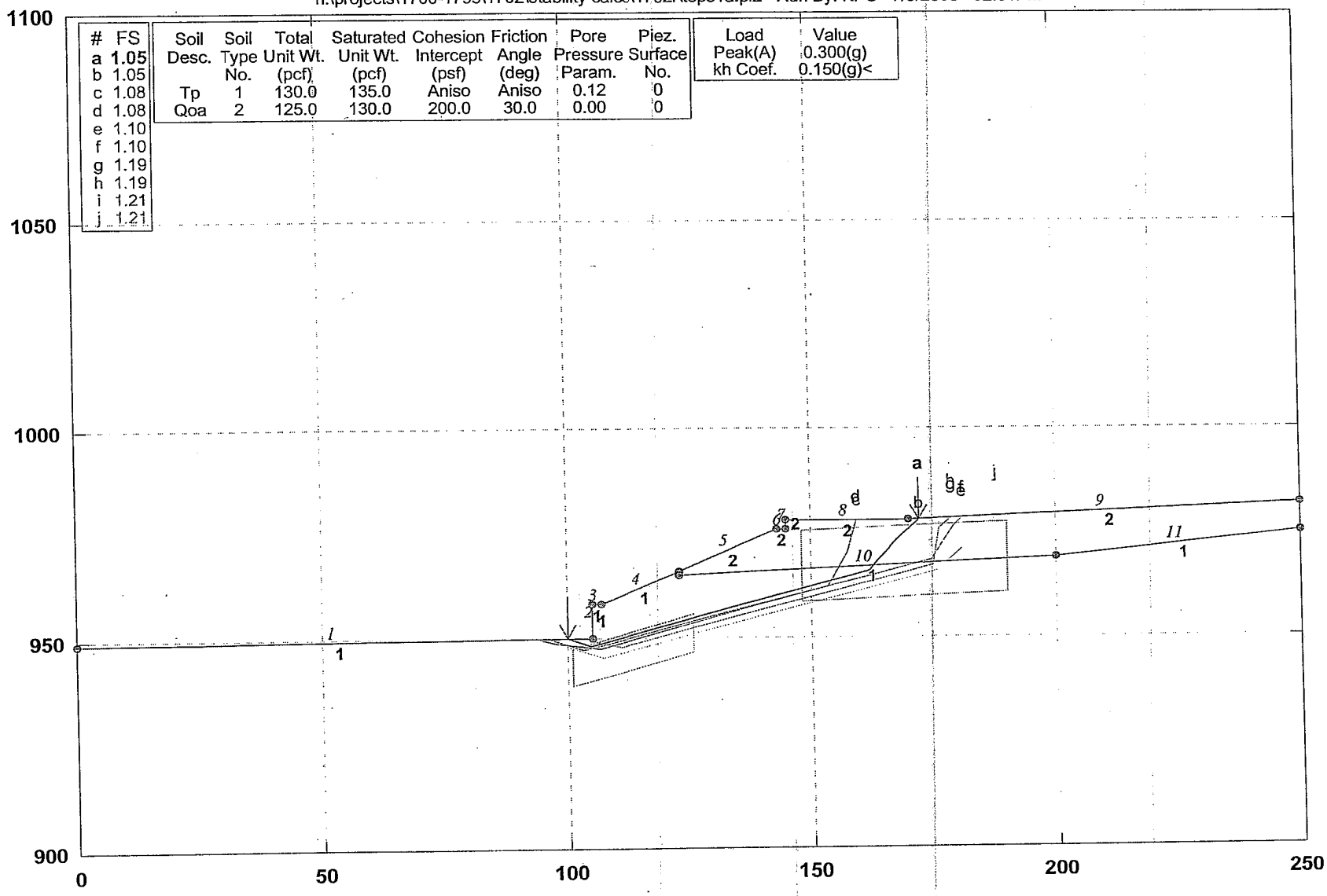
TT 53108, 3-3'; pseudo; planar; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t3ps1a.pl2 Run By: KPC 7/9/2008 02:51PM

#	FS
a	1.05
b	1.05
c	1.08
d	1.08
e	1.10
f	1.10
g	1.19
h	1.19
i	1.21
j	1.21

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Qoa	2	125.0	130.0	200.0	30.0	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.05

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 10

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                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                02:51PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t3ps1a.PLT
PROBLEM DESCRIPTION:  CT 53108, 3-3'; pseudo; planar;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    9 Top   Boundaries
   11 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1          0.00      949.00      105.00      950.00        1
    2        105.00      950.00      105.01      958.00        1
    3        105.01      958.00      107.00      958.00        1
    4        107.00      958.00      123.00      966.00        1
    5        123.00      966.00      143.00      976.00        2
    6        143.00      976.00      145.00      976.00        2
    7        145.00      976.00      145.01      978.00        2
    8        145.01      978.00      170.00      978.00        2
    9        170.00      978.00      250.00      982.00        2
   10        123.00      965.00      200.00      969.00        1
   11        200.00      969.00      250.00      975.00        1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle   Pressure Constant Surface
  No.   (pcf)    (pcf)     (psf)    (deg)    Param.    (psf)     No.
   1    130.0    135.0     203.0     45.0     0.12      0.0       0
   2    125.0    130.0     200.0     30.0     0.00      0.0       0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
 Soil Type  1 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction     Counterclockwise     Cohesion      Friction
   Range       Direction Limit     Intercept       Angle
    No.            (deg)             (psf)         (deg)
     1              15.5             203.00         45.00
     2              18.5             200.00         18.00
     3              90.0             203.00         45.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 Specified Peak Ground Acceleration Coefficient (A) =  0.300(g)
 Specified Horizontal Earthquake Coefficient (kh) =  0.150(g)
```

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
1000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 9.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	101.00	943.00	126.00	951.00	9.00
2	148.00	967.00	190.00	969.00	17.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 214
Number of Trial Surfaces with Misleading FS = 14
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 22.8 %
Statistical Data On All Valid FS Values:
FS Max = 6.754 FS Min = 1.052 FS Ave = 2.797
Standard Deviation = 0.676 Coefficient of Variation = 24.17 %
((Simplified Janbu FS for Critical Surface = 0.932))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	949.95
2	104.54	947.98
3	161.97	965.85
4	167.41	973.03
5	172.21	978.11

*** FOS = 1.052 Theta (Xi=1.0) = 15.48 ***
Lambda = 0.277
Individual data on the 13 slices

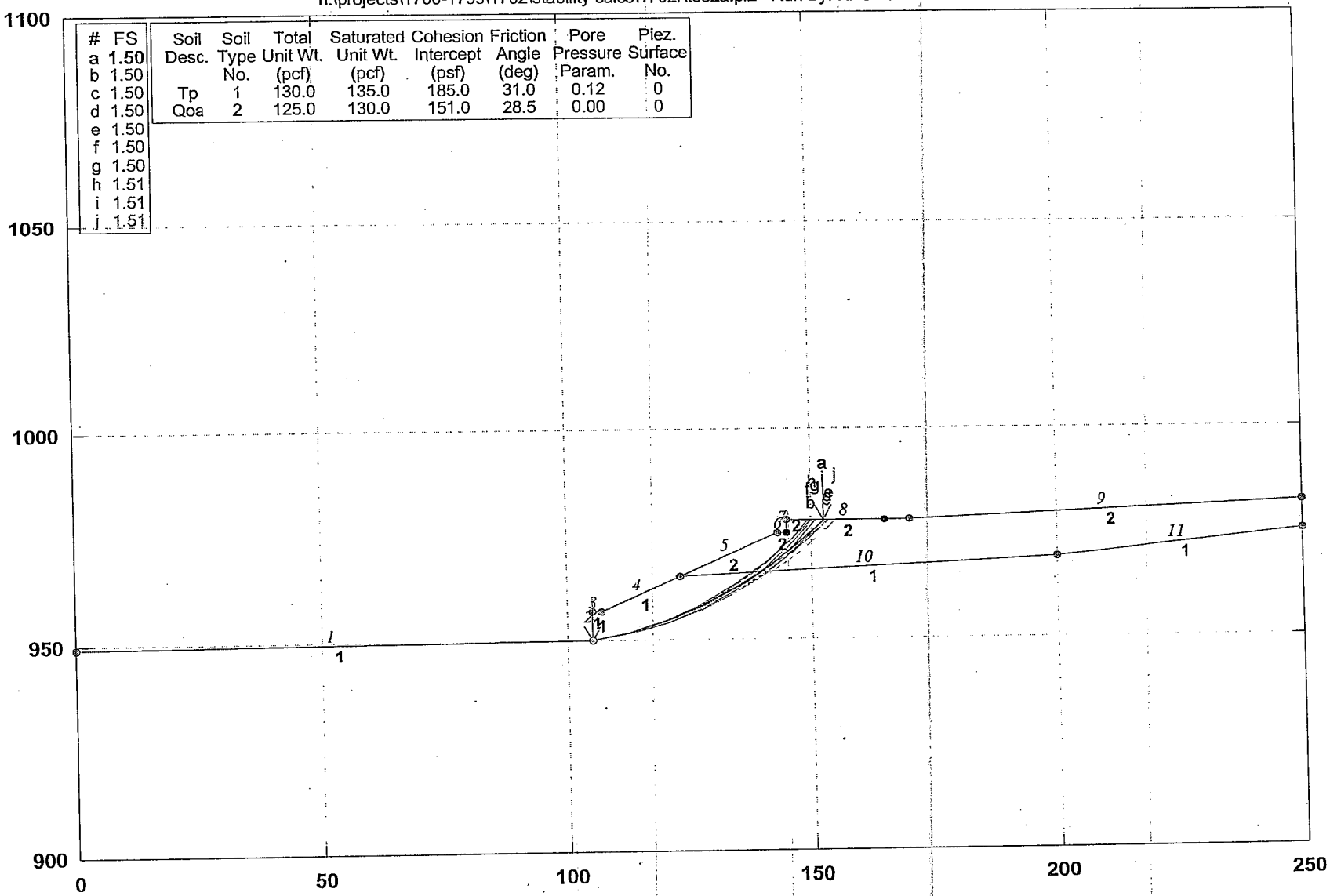
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.5	595.6	0.0	77.9	0.	0.	89.3	0.0	0.0
2	0.5	115.3	0.0	14.5	0.	0.	17.3	0.0	0.0
3	0.0	7.6	0.0	1.0	0.	0.	1.1	0.0	0.0
4	2.0	2474.8	0.0	311.0	0.	0.	371.2	0.0	0.0
5	16.0	22394.5	0.0	2814.5	0.	0.	3359.2	0.0	0.0
6	20.0	36277.7	0.0	4559.3	0.	0.	5441.7	0.0	0.0
7	2.0	3993.3	0.0	501.9	0.	0.	599.0	0.0	0.0
8	0.0	20.8	0.0	2.6	0.	0.	3.1	0.0	0.0
9	17.0	31637.2	0.0	3976.1	0.	0.	4745.6	0.0	0.0
10	0.9	1331.8	0.0	264.8	0.	0.	199.8	0.0	0.0
11	4.5	4480.9	0.0	0.0	0.	0.	672.1	0.0	0.0
12	2.6	1167.1	0.0	0.0	0.	0.	175.1	0.0	0.0
13	2.2	308.0	0.0	0.0	0.	0.	46.2	0.0	0.0

TT 53108, 3-3'; static; circular; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t3s2a.pl2 Run By: KPC 7/9/2008 02:19PM

#	FS
a	1.50
b	1.50
c	1.50
d	1.50
e	1.50
f	1.50
g	1.50
h	1.51
i	1.51
j	1.51

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	185.0	31.0	0.12	0
Qoa	2	125.0	130.0	151.0	28.5	0.00	0



GSTABL7 v.2 FSmin=1.50

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 11

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                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
  ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                02:19PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi-Calcs\1702R\t3s2a.PLT
PROBLEM DESCRIPTION:  TT 53108, 3-3'; static; circular;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    9 Top   Boundaries
   11 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)        (ft)        (ft)     Below Bnd
    1         0.00      949.00      105.00      950.00         1
    2       105.00      950.00      105.01      957.00         1
    3       105.01      957.00      107.00      957.00         1
    4       107.00      957.00      123.00      965.00         1
    5       123.00      965.00      143.00      975.00         2
    6       143.00      975.00      145.00      975.00         2
    7       145.00      975.00      145.01      978.00         2
    8       145.01      978.00      170.00      978.00         2
    9       170.00      978.00      250.00      982.00         2
   10       123.00      965.00      200.00      969.00         1
   11       200.00      969.00      250.00      975.00         1
 User Specified Y-Origin =       900.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
   1   130.0    135.0     165.0    31.0    0.12      0.0       0
   2   125.0    130.0     151.0    28.5    0.00      0.0       0
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
  750 Trial Surfaces Have Been Generated.
  750 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
 Along The Ground Surface Between  X = 105.00(ft)
                              and  X = 105.00(ft)
 Each Surface Terminates Between   X = 145.00(ft)
                              and  X = 165.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =       0.00(ft)
  8.00(ft) Line Segments Define Each Trial Failure Surface.
 Restrictions Have Been Imposed Upon The Angle Of Initiation.
 The Angle Has Been Restricted Between The Angles Of -20.0
 And  15.0 deg.
 Factor Of Safety Calculation By GLE (Spencer's) Method of Slices
 Has Gone Through 20 Iterations
  Theta        FOS         FOS
  (deg)      (Moment)    (Force)
 (ki=1.0)    (Equil.)    (Equil.)    Lambda
```

13.00	1.606	1.473	0.231
19.50	1.577	1.481	0.354
40.81	25.987	1.517	0.863
19.39	1.577	1.481	0.352
12.14	1.609	1.471	0.215
44.04	2.823	1.524	0.967
7.10	1.622	1.465	0.125
0.00	1.633	1.456	0.000
83.69	1.428	20.000	9.047
1.05	1.631	1.457	0.018
0.00	1.633	1.456	0.000
89.94	1.408	307.372	*****
0.07	1.633	1.456	0.001
0.00	1.633	1.456	0.000
89.94	1.408	0.434	*****
0.00	1.633	1.456	0.000
0.00	1.633	1.456	0.000
0.00	1.631	1.456	0.000
0.00	1.631	1.456	0.000
0.00	1.631	1.456	0.000

The Trial Failure Surface In Question Is Defined
By The Following 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	105.00	950.00
2	112.74	952.03
3	120.24	954.81
4	127.43	958.31
5	134.24	962.52
6	140.60	967.37
7	146.45	972.83
8	151.00	978.00

Following Are Displayed The Ten Most Critical Of The Trial
failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 750
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 1
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.1 %
Statistical Data On All Valid FS Values:
FS Max = 2.241 FS Min = 1.496 FS Ave = 1.905
Standard Deviation = 0.225 Coefficient of Variation = 11.81 %
((Modified Bishop FS for Critical Surface = 1.493))
Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	105.00	950.00
2	112.76	951.96
3	120.29	954.65
4	127.54	958.04
5	134.43	962.11
6	140.90	966.81
7	146.90	972.10
8	152.36	977.94
9	152.41	978.00

Circle Center At X = 88.26 ; Y = 1032.50 ; and Radius = 84.18
*** FOS = 1.496 Theta (ki=1.0) = 27.12 ***

Lambda = 0.512

Individual data on the 15 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	4.6	0.0	0.6	0.	0.	0.0	0.0	0.0
2	2.0	1745.1	0.0	216.0	0.	0.	0.0	0.0	0.0
3	5.8	5391.0	0.0	667.3	0.	0.	0.0	0.0	0.0
4	7.5	8281.0	0.0	1055.1	0.	0.	0.0	0.0	0.0
5	2.7	3183.5	0.0	421.8	0.	0.	0.0	0.0	0.0
6	4.5	5372.3	0.0	711.9	0.	0.	0.0	0.0	0.0
7	6.9	7860.9	0.0	1095.2	0.	0.	0.0	0.0	0.0
8	5.2	5225.3	0.0	774.9	0.	0.	0.0	0.0	0.0
9	1.3	1189.0	0.0	0.0	0.	0.	0.0	0.0	0.0
10	2.1	1769.7	0.0	0.0	0.	0.	0.0	0.0	0.0
11	2.0	1363.2	0.0	0.0	0.	0.	0.0	0.0	0.0
12	0.0	7.6	0.0	0.0	0.	0.	0.0	0.0	0.0
13	1.9	1586.6	0.0	0.0	0.	0.	0.0	0.0	0.0
14	5.5	2034.4	0.0	0.0	0.	0.	0.0	0.0	0.0
15	0.0	0.2	0.0	0.0	0.	0.	0.0	0.0	0.0

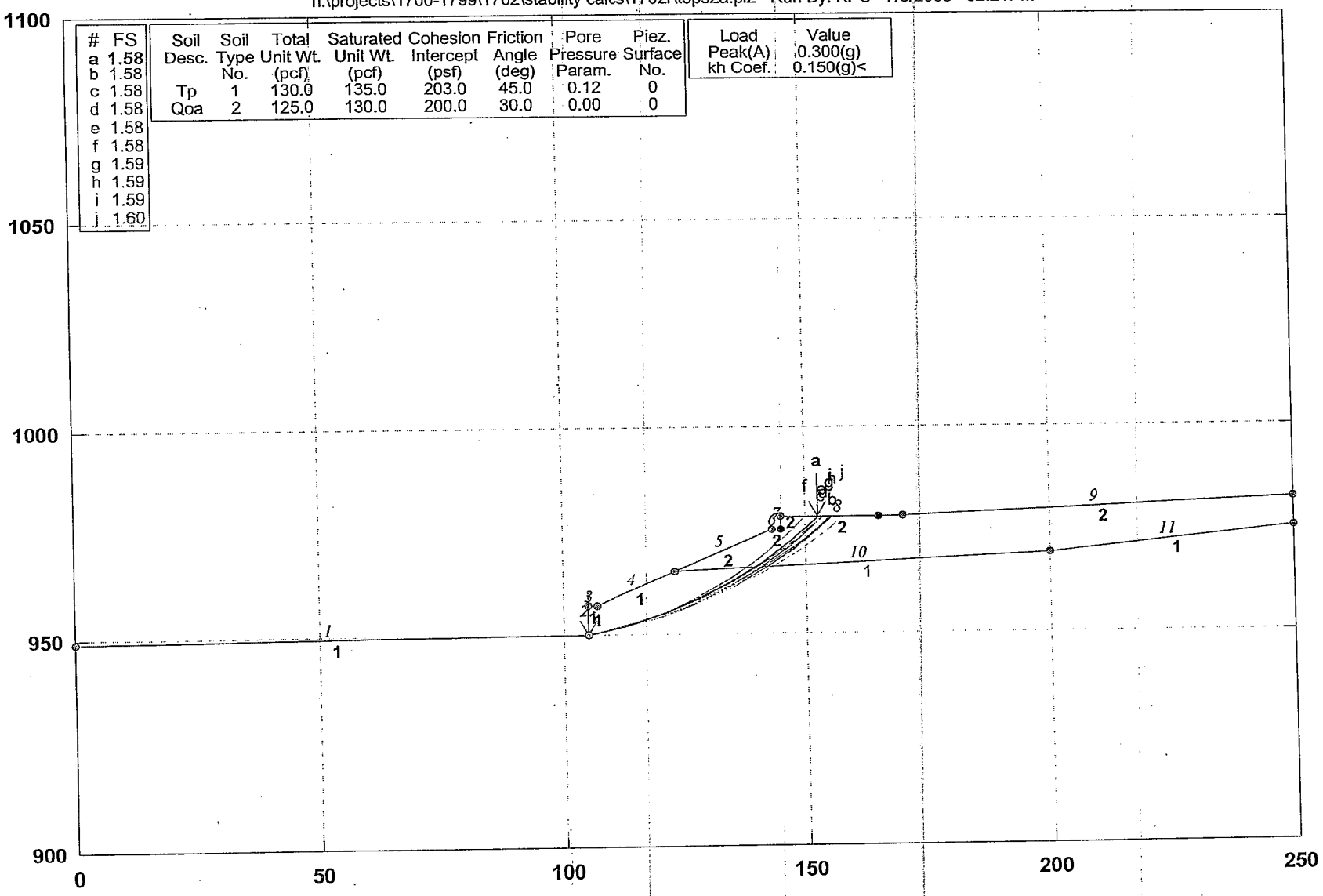
TT 53108, 3-3'; pseudo; circular; daylighted bedding w/ Ru=0.12

n:\projects\1700-1799\1702\stability calcs\1702r\t3ps2a.pl2 Run By: KPC 7/9/2008 02:21PM

#	FS
a	1.58
b	1.58
c	1.58
d	1.58
e	1.58
f	1.58
g	1.59
h	1.59
i	1.59
j	1.60

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	203.0	45.0	0.12	0
Qoa	2	125.0	130.0	200.0	30.0	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.58

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 12

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                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                02:21PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t3ps2a.PLT
PROBLEM DESCRIPTION:  TT 53108, 3-3'; pseudo; circular;
                      daylighted bedding w/ Ru=0.12

BOUNDARY COORDINATES
    9 Top   Boundaries
   11 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)     Below Bnd
    1          0.00      949.00     105.00      950.00        1
    2        105.00      950.00     105.01      957.00        1
    3        105.01      957.00     107.00      957.00        1
    4        107.00      957.00     123.00      965.00        1
    5        123.00      965.00     143.00      975.00        2
    6        143.00      975.00     145.00      975.00        2
    7        145.00      975.00     145.01      978.00        2
    8        145.01      978.00     170.00      978.00        2
    9        170.00      978.00     250.00      982.00        2
   10        123.00      965.00     200.00      969.00        1
   11        200.00      969.00     250.00      975.00        1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
 2 Type(s) of Soil
Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
  1   130.0    135.0     203.0      45.0    0.12       0.0      0
  2   125.0    130.0     200.0      30.0    0.00       0.0      0
Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
Specified Seismic Pore-Pressure Factor =   0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
 750 Trial Surfaces Have Been Generated.
 750 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
Along The Ground Surface Between  X = 105.00(ft)
                            and   X = 105.00(ft)
Each Surface Terminates Between   X = 145.00(ft)
                            and   X = 165.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =       0.00(ft)
 8.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -20.0
And  15.0 deg.
```

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 750
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 9
Number of Trial Surfaces with Misleading FS = 2
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 1.5 %
Statistical Data On All Valid FS Values:
FS Max = 2.361 FS Min = 1.575 FS Ave = 2.028
Standard Deviation = 0.232 Coefficient of Variation = 11.45 %
((Modified Bishop FS for Critical Surface = 1.580))
Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	105.00	950.00
2	112.76	951.96
3	120.29	954.65
4	127.54	958.04
5	134.43	962.11
6	140.90	966.81
7	146.90	972.10
8	152.36	977.94
9	152.41	978.00

Circle Center At X = 88.26 ; Y = 1032.50 ; and Radius = 84.18
*** FOS = 1.575 Theta (ki=1.0) = 27.56 ***
Lambda = 0.522
Individual data on the 15 slices

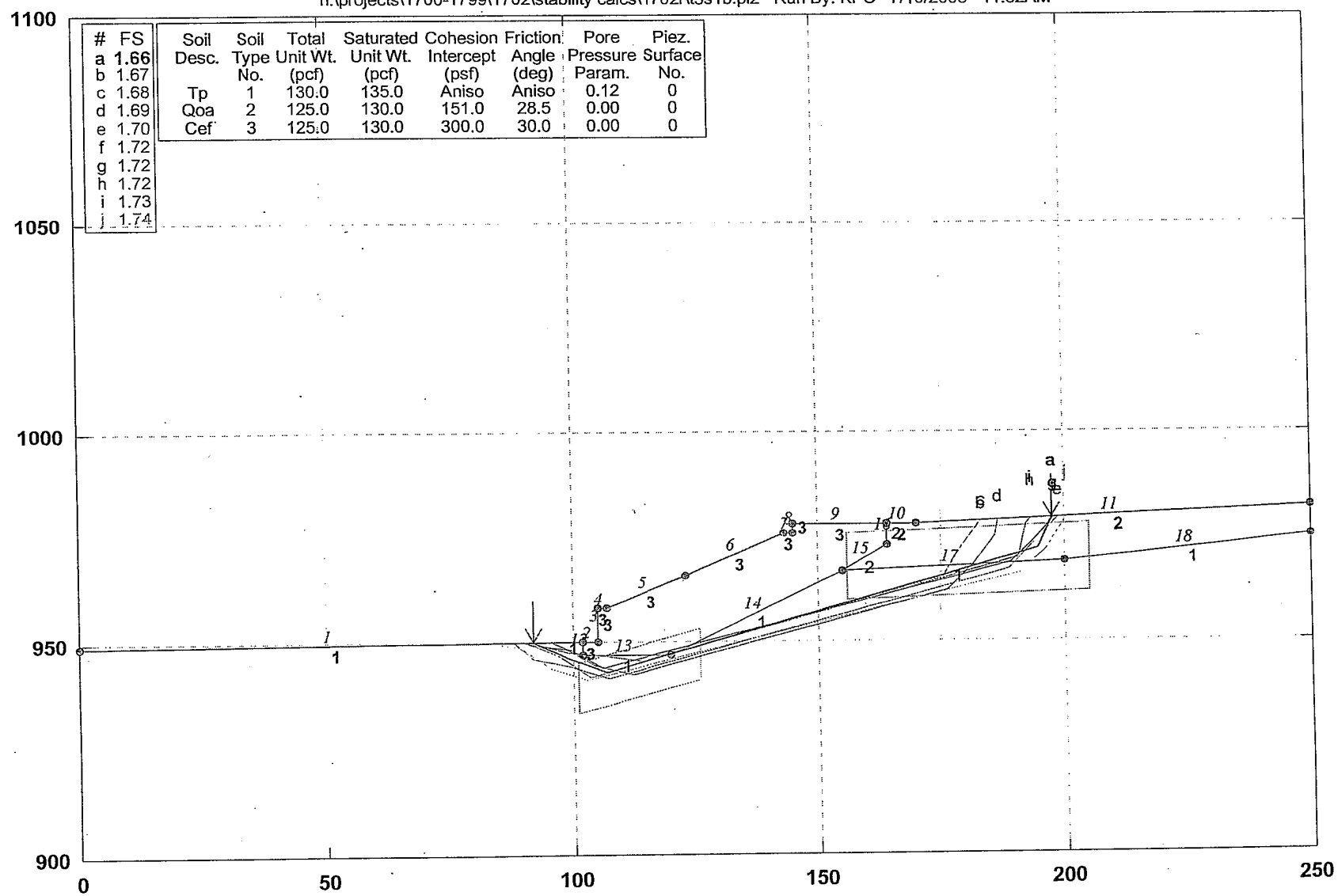
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.0	4.6	0.0	0.6	0.	0.	0.7	0.0	0.0
2	2.0	1745.1	0.0	216.0	0.	0.	261.8	0.0	0.0
3	5.8	5391.0	0.0	667.3	0.	0.	808.6	0.0	0.0
4	7.5	8281.0	0.0	1055.1	0.	0.	1242.2	0.0	0.0
5	2.7	3183.5	0.0	421.8	0.	0.	477.5	0.0	0.0
6	4.5	5372.3	0.0	711.9	0.	0.	805.8	0.0	0.0
7	6.9	7860.9	0.0	1095.2	0.	0.	1179.1	0.0	0.0
8	5.2	5225.3	0.0	774.9	0.	0.	783.8	0.0	0.0
9	1.3	1189.0	0.0	0.0	0.	0.	178.3	0.0	0.0
10	2.1	1769.7	0.0	0.0	0.	0.	265.5	0.0	0.0
11	2.0	1363.2	0.0	0.0	0.	0.	204.5	0.0	0.0
12	0.0	7.6	0.0	0.0	0.	0.	1.1	0.0	0.0
13	1.9	1586.6	0.0	0.0	0.	0.	238.0	0.0	0.0
14	5.5	2034.4	0.0	0.0	0.	0.	305.2	0.0	0.0
15	0.0	0.2	0.0	0.0	0.	0.	0.0	0.0	0.0

TT 53108, 3-3'; static; planar; Ru=0.12; stab fill w/ 3' x 15' key

n:\projects\1700-1799\1702\stability calcs\1702r\t3s1b.pl2 Run By: KPC 7/10/2008 11:32AM

#	FS
a	1.66
b	1.67
c	1.68
d	1.69
e	1.70
f	1.72
g	1.72
h	1.72
i	1.73
j	1.74

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Qoa	2	125.0	130.0	151.0	28.5	0.00	0
Cef	3	125.0	130.0	300.0	30.0	0.00	0



GSTABL7 v.2 FSmin=1.66

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 13

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                11:32AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3s1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t3s1b.PLT
PROBLEM DESCRIPTION:  TT 53108, 3-3'; static; planar;
                      Ru=0.12; stab fill w/ 3' x 15' key

BOUNDARY COORDINATES
   11 Top   Boundaries
   18 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1          0.00      949.00     102.00      950.00         1
    2        102.00      950.00     105.00      950.00         3
    3        105.00      950.00     105.01      958.00         3
    4        105.01      958.00     107.00      958.00         3
    5        107.00      958.00     123.00      966.00         3
    6        123.00      966.00     143.00      976.00         3
    7        143.00      976.00     145.00      976.00         3
    8        145.00      976.00     145.01      978.00         3
    9        145.01      978.00     164.00      978.00         3
   10        164.00      978.00     170.00      978.00         2
   11        170.00      978.00     250.00      982.00         2
   12        102.00      950.00     102.01      947.00         1
   13        102.01      947.00     120.00      947.00         1
   14        120.00      947.00     155.00      967.00         1
   15        155.00      967.00     163.99      973.00         2
   16        163.99      973.00     164.00      978.00         2
   17        155.00      967.00     200.00      969.00         1
   18        200.00      969.00     250.00      975.00         1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  3 Type(s) of Soil
Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle   Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)    Param.    (psf)     No.
  1   130.0    135.0     185.0     31.0     0.12      0.0       0
  2   125.0    130.0     151.0     28.5     0.00      0.0       0
  3   125.0    130.0     300.0     30.0     0.00      0.0       0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
Soil Type  1 Is Anisotropic
Number Of Direction Ranges Specified =  3
Direction     Counterclockwise     Cohesion      Friction
  Range       Direction Limit     Intercept       Angle
   No.            (deg)             (psf)         (deg)
    1              15.5             185.00         31.00
    2              18.5             200.00         18.00
    3              90.0             185.00         31.00
ANISOTROPIC SOIL NOTES:
```

(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

750 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 6.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	101.00	939.00	126.00	947.00	12.00
2	156.00	968.00	205.00	970.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *

Selected ki function = Bi-linear

Selected Lambda Coefficient = 1.00

The option of no water in the tension crack on the last slice has been selected.

Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.

Total Number of Trial Surfaces Evaluated = 750

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 7

Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 0.9 %

Statistical Data On All Valid FS Values:

FS Max = 6.536 FS Min = 1.655 FS Ave = 2.853

Standard Deviation = 0.722 Coefficient of Variation = 25.30 %

((Simplified Janbu FS for Critical Surface = 1.445))

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.72	949.90
2	92.19	949.55
3	99.39	946.06
4	106.77	942.96
5	194.65	972.23
6	197.42	979.37

*** FOS = 1.655 Theta (ki=1.0) = 12.89 ***

Lambda = 0.229

Individual data on the 21 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.5	10.8	0.0	1.6	0.	0.	0.0	0.0	0.0
2	7.2	1995.7	0.0	266.2	0.	0.	0.0	0.0	0.0
3	2.6	1517.6	0.0	197.6	0.	0.	0.0	0.0	0.0
4	0.0	6.5	0.0	0.8	0.	0.	0.0	0.0	0.0
5	3.0	2158.8	0.0	281.0	0.	0.	0.0	0.0	0.0
6	0.0	13.0	0.0	1.7	0.	0.	0.0	0.0	0.0
7	1.8	3253.1	0.0	423.5	0.	0.	0.0	0.0	0.0
8	0.2	442.9	0.0	56.0	0.	0.	0.0	0.0	0.0
9	11.9	23843.2	0.0	3015.7	0.	0.	0.0	0.0	0.0
10	2.6	5681.0	0.0	0.0	0.	0.	0.0	0.0	0.0
11	1.5	3190.5	0.0	403.5	0.	0.	0.0	0.0	0.0
12	20.0	48528.2	0.0	6137.9	0.	0.	0.0	0.0	0.0
13	2.0	5213.0	0.0	659.4	0.	0.	0.0	0.0	0.0

14	0.0	26.9	0.0	3.4	0.	0.	0.0	0.0	0.0
15	10.0	26110.3	0.0	3302.5	0.	0.	0.0	0.0	0.0
16	9.0	19939.6	0.0	2522.0	0.	0.	0.0	0.0	0.0
17	0.0	20.3	0.0	2.6	0.	0.	0.0	0.0	0.0
18	6.0	11368.0	0.0	1437.8	0.	0.	0.0	0.0	0.0
19	12.6	19353.9	0.0	2447.9	0.	0.	0.0	0.0	0.0
20	12.0	13078.2	0.0	0.0	0.	0.	0.0	0.0	0.0
21	2.8	1210.3	0.0	0.0	0.	0.	0.0	0.0	0.0
	***								

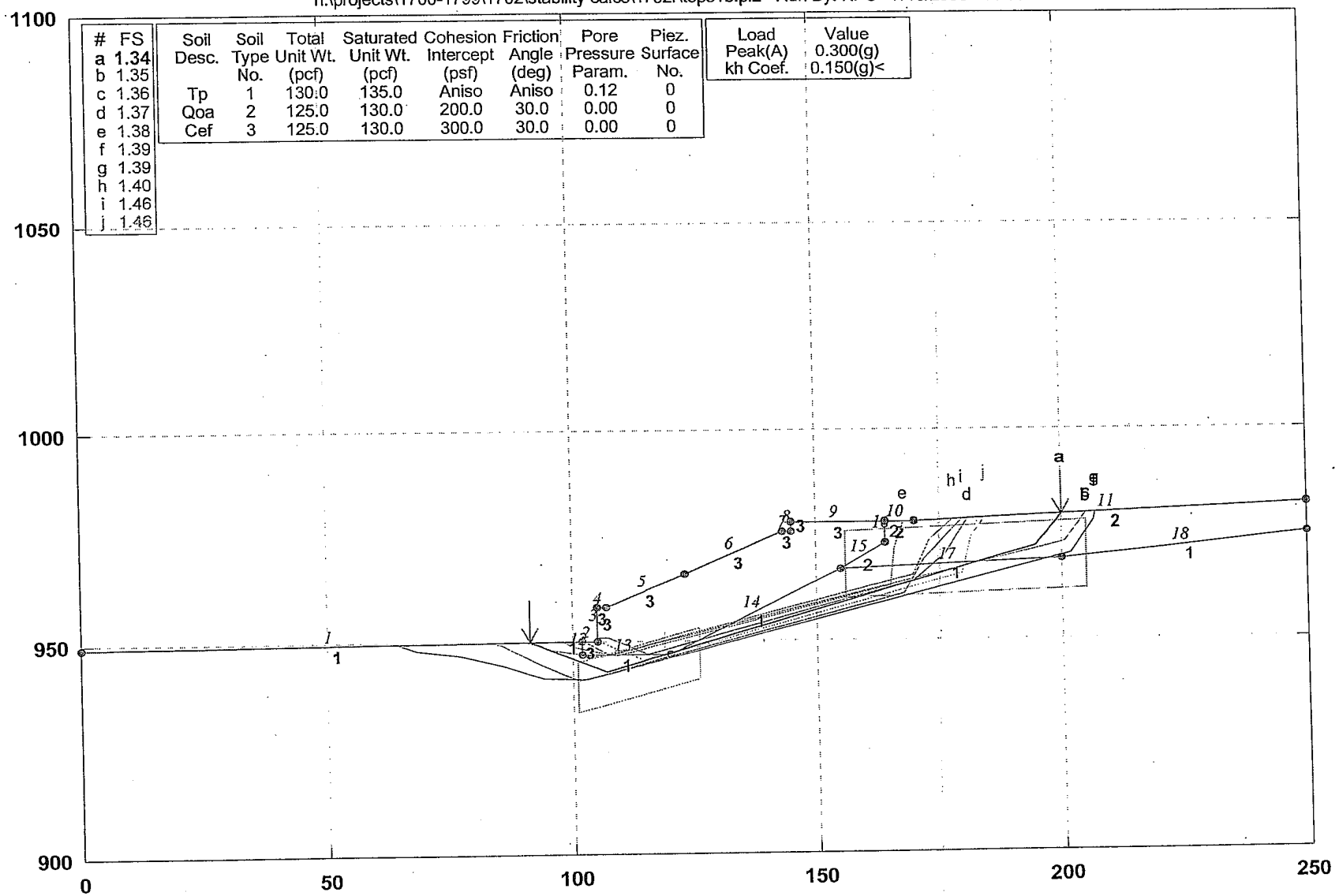
TT 53108, 3-3'; pseudo; planar; Ru=0.12; stab fill w/ 3' x 15' key

n:\projects\1700-1799\1702\stability calcs\1702r\t3ps1b.pl2 Run By: KPC 7/10/2008 11:33AM

#	FS
a	1.34
b	1.35
c	1.36
d	1.37
e	1.38
f	1.39
g	1.39
h	1.40
i	1.46
j	1.46

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Tp	1	130.0	135.0	Aniso	Aniso	0.12	0
Qoa	2	125.0	130.0	200.0	30.0	0.00	0
Cef	3	125.0	130.0	300.0	30.0	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.34

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No: 14

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                11:33AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t3ps1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t3ps1b.PLT
PROBLEM DESCRIPTION:  CT 53108, 3-3'; pseudo; planar;
                      Ru=0.12; stab fill w/ 3' x 15' key

BOUNDARY COORDINATES
   11 Top   Boundaries
   18 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1         0.00      949.00      102.00      950.00         1
    2       102.00      950.00      105.00      950.00         3
    3       105.00      950.00      105.01      958.00         3
    4       105.01      958.00      107.00      958.00         3
    5       107.00      958.00      123.00      966.00         3
    6       123.00      966.00      143.00      976.00         3
    7       143.00      976.00      145.00      976.00         3
    8       145.00      976.00      145.01      978.00         3
    9       145.01      978.00      164.00      978.00         3
   10       164.00      978.00      170.00      978.00         2
   11       170.00      978.00      250.00      982.00         2
   12       102.00      950.00      102.01      947.00         1
   13       102.01      947.00      120.00      947.00         1
   14       120.00      947.00      155.00      967.00         1
   15       155.00      967.00      163.99      973.00         2
   16       163.99      973.00      164.00      978.00         2
   17       155.00      967.00      200.00      969.00         1
   18       200.00      969.00      250.00      975.00         1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  3 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept  Angle   Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)    (deg)    Param.    (psf)     No.
  1   130.0    135.0     203.0     45.0     0.12      0.0       0
  2   125.0    130.0     200.0     30.0     0.00      0.0       0
  3   125.0    130.0     300.0     30.0     0.00      0.0       0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
Soil Type  1 Is Anisotropic
Number Of Direction Ranges Specified =  3
Direction     Counterclockwise     Cohesion      Friction
  Range       Direction Limit     Intercept       Angle
   No.            (deg)             (psf)         (deg)
    1              15.5             203.00         45.00
    2              18.5             200.00         18.00
    3              90.0             203.00         45.00
ANISOTROPIC SOIL NOTES:
```

(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
750 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 9.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	101.00	939.00	126.00	947.00	12.00
2	156.00	968.00	205.00	970.00	16.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 750
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 153
Number of Trial Surfaces with Misleading FS = 12
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 22.0 %
Statistical Data On All Valid FS Values:
FS Max = 5.687 FS Min = 1.337 FS Ave = 2.675
Standard Deviation = 0.601 Coefficient of Variation = 22.45 %
[(Simplified Janbu FS for Critical Surface = 1.152))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	91.36	949.90
2	98.47	946.45
3	106.77	942.96
4	194.65	972.23
5	199.98	979.49
6	199.98	979.50

*** FOS = 1.337 Theta (ki=1.0) = 12.87 ***
Lambda = 0.229
Individual data on the 21 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	7.1	1624.8	0.0	216.7	0.	0.	243.7	0.0	0.0
2	3.5	1962.9	0.0	255.5	0.	0.	294.4	0.0	0.0
3	0.0	6.5	0.0	0.8	0.	0.	1.0	0.0	0.0
4	3.0	2158.8	0.0	281.0	0.	0.	323.8	0.0	0.0
5	0.0	13.0	0.0	1.7	0.	0.	2.0	0.0	0.0
6	1.8	3253.1	0.0	423.5	0.	0.	488.0	0.0	0.0
7	0.2	442.9	0.0	56.0	0.	0.	66.4	0.0	0.0

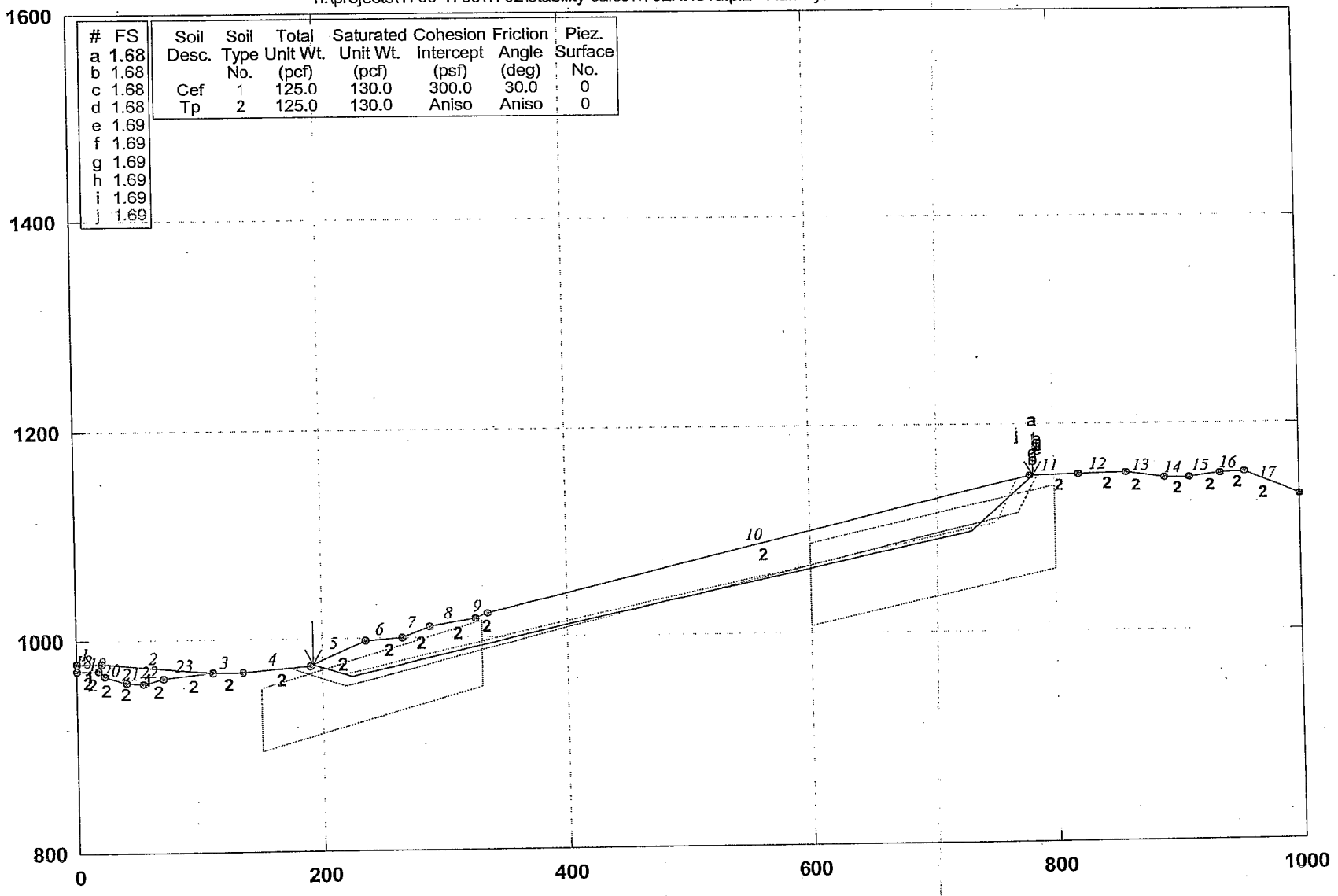
8	11.9	23843.1	0.0	3015.7	0.	0.	3576.5	0.0	0.0
9	2.6	5681.0	0.0	0.0	0.	0.	852.1	0.0	0.0
10	1.5	3190.5	0.0	403.5	0.	0.	478.6	0.0	0.0
11	20.0	48528.2	0.0	6137.9	0.	0.	7279.2	0.0	0.0
12	2.0	5213.0	0.0	659.4	0.	0.	782.0	0.0	0.0
13	0.0	26.9	0.0	3.4	0.	0.	4.0	0.0	0.0
14	10.0	26110.3	0.0	3302.5	0.	0.	3916.5	0.0	0.0
15	9.0	19939.6	0.0	2522.0	0.	0.	2990.9	0.0	0.0
16	0.0	20.3	0.0	2.6	0.	0.	3.0	0.0	0.0
17	6.0	11368.0	0.0	1437.8	0.	0.	1705.2	0.0	0.0
18	12.6	19353.9	0.0	2447.9	0.	0.	2903.1	0.0	0.0
19	12.0	13078.2	0.0	0.0	0.	0.	1961.7	0.0	0.0
20	5.3	2333.6	0.0	0.0	0.	0.	350.0	0.0	0.0
21	0.0	0.0	0.0	0.0	0.	0.	0.0	0.0	0.0

Landmark off-site grading; 4-4'; CS-2; static; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t4s1a.pl2 Run By: KPC 7/10/2008 09:51AM

#	FS
a	1.68
b	1.68
c	1.68
d	1.68
e	1.69
f	1.69
g	1.69
h	1.69
i	1.69
j	1.69

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0
Tp	2	125.0	130.0	Aniso	Aniso	0



GSTABL7 v.2 FSmin=1.68

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 15

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                09:51AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t4s1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t4s1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t4s1a.PLT
PROBLEM DESCRIPTION:  Landmark off-site grading; 4-4'; CS-2;
                      static; planar

BOUNDARY COORDINATES
   17 Top   Boundaries
   23 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1         0.00      979.00      20.00       979.00        1
    2        20.00      979.00     110.00       970.00        1
    3       110.00      970.00     135.00       970.00        2
    4       135.00      970.00     190.00       975.00        2
    5       190.00      975.00     235.00       998.00        2
    6       235.00      998.00     255.00      1000.00        2
    7       255.00     1000.00     288.00      1010.00        2
    8       288.00     1010.00     325.00      1017.00        2
    9       325.00     1017.00     335.00      1022.00        2
   10       335.00     1022.00     780.00      1150.00        2
   11       780.00     1150.00     820.00      1151.00        2
   12       820.00     1151.00     859.00      1153.00        2
   13       859.00     1153.00     890.00      1148.00        2
   14       890.00     1148.00     910.00      1148.00        2
   15       910.00     1148.00     935.00      1151.00        2
   16       935.00     1151.00     955.00      1153.00        2
   17       955.00     1153.00    1000.00      1130.00        2
   18         0.00      973.00      17.00       972.00        2
   19        17.00      972.00      23.00       967.00        2
   20        23.00      967.00      40.00       961.00        2
   21        40.00      961.00      54.00       960.00        2
   22        54.00      960.00      70.00       965.00        2
   23        70.00      965.00     110.00       970.00        2
 User Specified Y-Origin =        800.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1   125.0    130.0     300.0     30.0    0.00       0.0      0
   2   125.0    130.0     350.0     30.0    0.00       0.0      0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
 Soil Type  2 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction     Counterclockwise      Cohesion      Friction
   Range       Direction Limit      Intercept       Angle
    No.            (deg)              (psf)         (deg)
```

1	13.5	350.00	30.00
2	16.5	200.00	20.00
3	90.0	350.00	30.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
2000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	150.00	925.00	330.00	985.00	60.00
2	600.00	1046.00	800.00	1100.00	80.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 2000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 46
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 2.3 %
Statistical Data On All Valid FS Values:
FS Max = 5.018 FS Min = 1.681 FS Ave = 2.688
Standard Deviation = 0.538 Coefficient of Variation = 20.01 %
((Simplified Janbu FS for Critical Surface = 1.607))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	192.50	976.28
2	223.86	965.35
3	731.19	1096.57
4	766.19	1132.28
5	783.08	1150.08

*** FOS = 1.681 Theta (ki=1.0) = 17.55 ***
Lambda = 0.316
Individual data on the 10 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	31.4	52839.8	0.0	0.0	0.	0.	0.0	0.0	0.0
2	11.1	39492.6	0.0	0.0	0.	0.	0.0	0.0	0.0
3	30.0	100837.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	23.0	74852.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	37.0	123838.4	0.0	0.0	0.	0.	0.0	0.0	0.0
6	10.0	33371.8	0.0	0.0	0.	0.	0.0	0.0	0.0
7	396.2	1666265.5	0.0	0.0	0.	0.	0.0	0.0	0.0
8	35.0	116237.7	0.0	0.0	0.	0.	0.0	0.0	0.0
9	13.8	14603.5	0.0	0.0	0.	0.	0.0	0.0	0.0
10	3.1	610.7	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	192.50	976.28
2	223.86	965.35
3	731.19	1096.57
4	766.19	1132.28
5	783.08	1150.08

*** FOS = 1.681 Theta (ki=1.0) = 17.55 ***

Lambda = 0.316

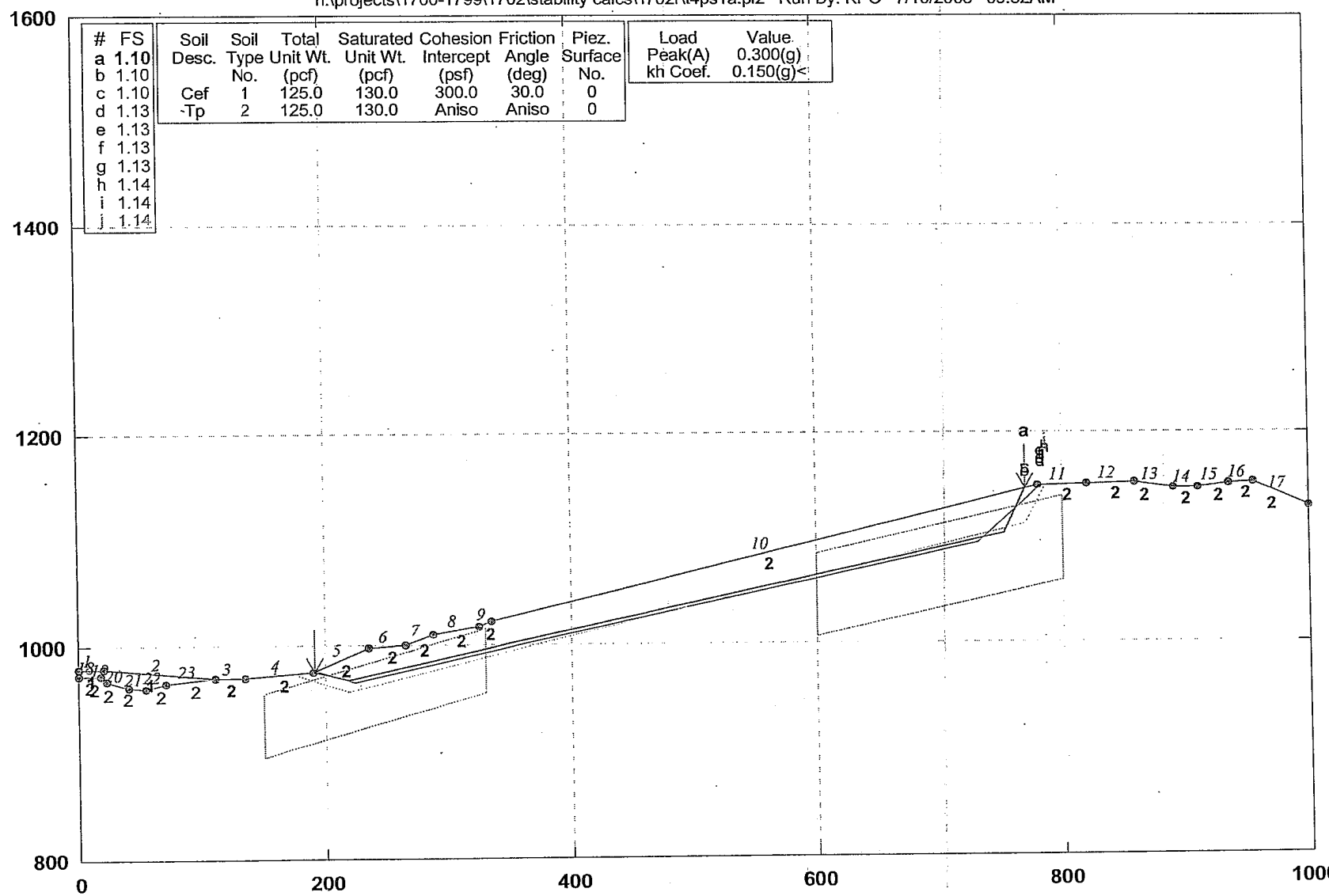
Landmark off-site grading; 4-4'; CS-2; pseudo; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t4ps1a.pl2 Run By: KPC 7/10/2008 09:52AM

#	FS
a	1.10
b	1.10
c	1.10
d	1.13
e	1.13
f	1.13
g	1.13
h	1.14
i	1.14
j	1.14

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0
-Tp	2	125.0	130.0	Aniso	Aniso	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<=



GSTABL7 v.2 FSmin=1.10

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 16

```
                    *** GSTABL7 ***
               ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                09:52AM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t4psla.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t4psla.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t4psla.PLT
PROBLEM DESCRIPTION:  Landmark off-site grading; 4-4'; CS-2;
                      pseudo; planar
BOUNDARY COORDINATES
   17 Top   Boundaries
   23 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)     Below Bnd
    1          0.00      979.00      20.00      979.00        1
    2         20.00      979.00     110.00      970.00        1
    3        110.00      970.00     135.00      970.00        2
    4        135.00      970.00     190.00      975.00        2
    5        190.00      975.00     235.00      998.00        2
    6        235.00      998.00     265.00     1000.00        2
    7        265.00     1000.00     288.00     1010.00        2
    8        288.00     1010.00     325.00     1017.00        2
    9        325.00     1017.00     335.00     1022.00        2
   10        335.00     1022.00     780.00     1150.00        2
   11        780.00     1150.00     820.00     1151.00        2
   12        820.00     1151.00     859.00     1153.00        2
   13        859.00     1153.00     890.00     1148.00        2
   14        890.00     1148.00     910.00     1148.00        2
   15        910.00     1148.00     935.00     1151.00        2
   16        935.00     1151.00     955.00     1153.00        2
   17        955.00     1153.00    1000.00     1130.00        2
   18          0.00      973.00      17.00      972.00        2
   19         17.00      972.00      23.00      967.00        2
   20         23.00      967.00      40.00      961.00        2
   21         40.00      961.00      54.00      960.00        2
   22         54.00      960.00      70.00      965.00        2
   23         70.00      965.00     110.00      970.00        2
 User Specified Y-Origin =       800.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.   (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1    125.0    130.0     300.0     30.0    0.00       0.0      0
   2    125.0    130.0     500.0     36.0    0.00       0.0      0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
 Soil Type  2 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction     Counterclockwise      Cohesion      Friction
   Range       Direction Limit     Intercept       Angle
    No.            (deg)             (psf)         (deg)
```

1	13.5	500.00	36.00
2	16.5	200.00	20.00
3	90.0	500.00	36.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
2000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	150.00	925.00	330.00	985.00	60.00
2	600.00	1046.00	800.00	1100.00	80.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 2000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 221
Number of Trial Surfaces with Misleading FS = 17
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 11.9 %
Statistical Data On All Valid FS Values:
FS Max = 3.149 FS Min = 1.103 FS Ave = 2.108
Standard Deviation = 0.435 Coefficient of Variation = 20.61 %
((Simplified Janbu FS for Critical Surface = 1.070))
Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	190.87	975.44
2	219.11	967.58
3	752.48	1105.52
4	770.61	1147.30

*** FOS = 1.103 Theta (ki=1.0) = 14.97 ***
Lambda = 0.267
Individual data on the 8 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	28.2	39378.7	0.0	0.0	0.	0.	5906.8	0.0	0.0
2	15.9	48271.7	0.0	0.0	0.	0.	7240.7	0.0	0.0
3	30.0	87886.0	0.0	0.0	0.	0.	13182.9	0.0	0.0
4	23.0	64924.4	0.0	0.0	0.	0.	9738.7	0.0	0.0
5	37.0	107870.0	0.0	0.0	0.	0.	16180.5	0.0	0.0

6	10.9	29056.5	0.0	0.0	0.	0.	4358.5	0.0	0.0
7	417.5	1591947.1	0.0	0.0	0.	0.	*******	0.0	0.0
8	18.1	41429.2	0.0	0.0	0.	0.	6214.4	0.0	0.0

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	190.87	975.44
2	219.11	967.58
3	752.48	1105.52
4	770.61	1147.30

*** FOS = 1.103 Theta (ki=1.0) = 14.97 ***

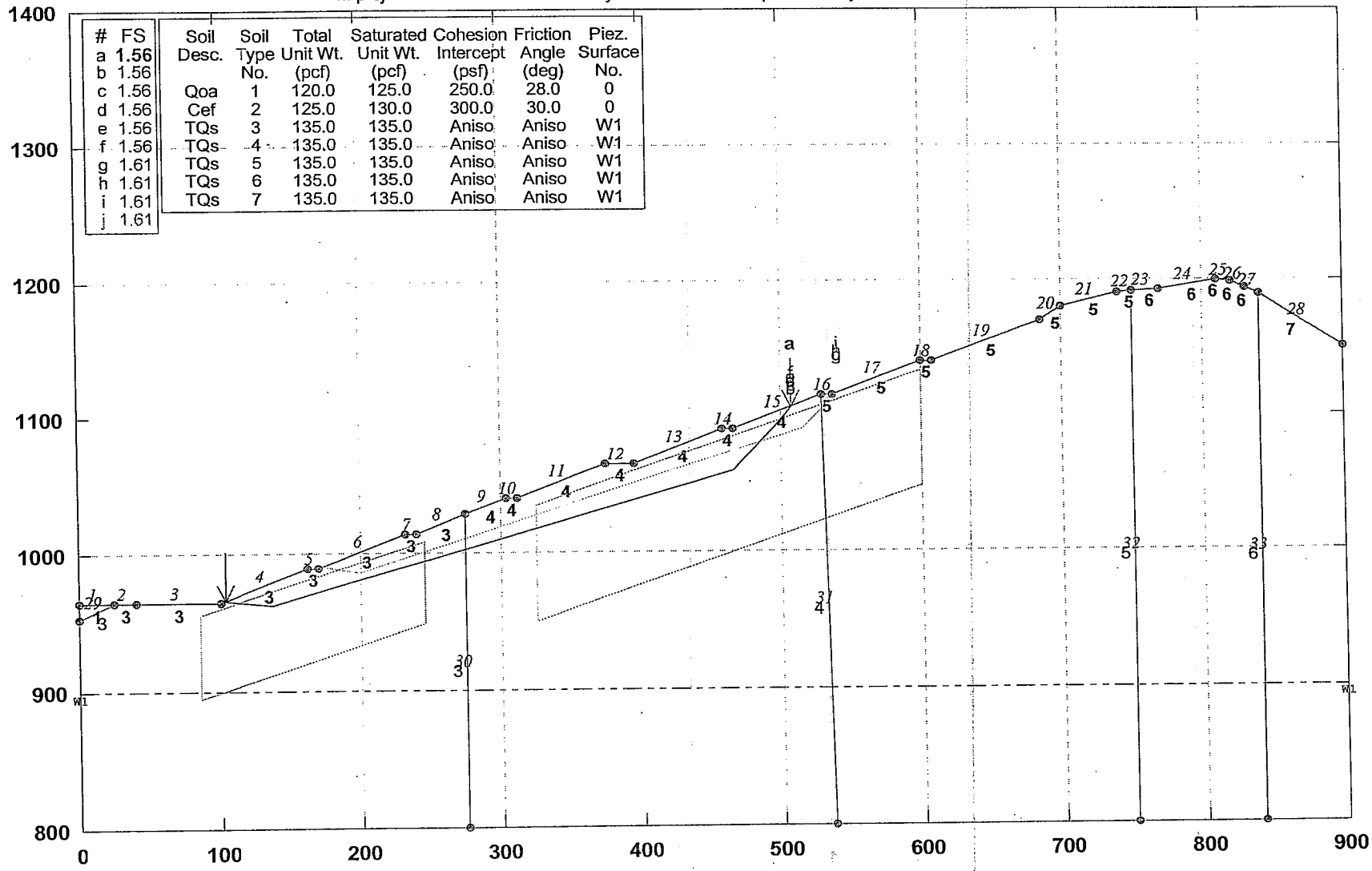
Lambda = 0.267

Lndmrk off-site grading; 5-5'; CS-4; static; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t5s2b.pl2 Run By: KPC 7/9/2008 10:29AM

#	FS
a	1.56
b	1.56
c	1.56
d	1.56
e	1.56
f	1.56
g	1.61
h	1.61
i	1.61
j	1.61

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qoa	1	120.0	125.0	250.0	28.0	0
Cef	2	125.0	130.0	300.0	30.0	0
TQs	3	135.0	135.0	Aniso	Aniso	W1
TQs	4	135.0	135.0	Aniso	Aniso	W1
TQs	5	135.0	135.0	Aniso	Aniso	W1
TQs	6	135.0	135.0	Aniso	Aniso	W1
TQs	7	135.0	135.0	Aniso	Aniso	W1



GSTABL7 v.2 FSmin=1.56

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 17

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 7/9/2008
Time of Run: 10:29AM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5s2b.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5s2b.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t5s2b.PLT
PROBLEM DESCRIPTION: Lndmrk off-site grading; 5-5'; CS-4; static; planar

BOUNDARY COORDINATES
28 Top Boundaries
33 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	964.00	25.00	964.00	1
2	25.00	964.00	40.00	964.00	3
3	40.00	964.00	100.00	964.00	3
4	100.00	964.00	162.00	989.00	3
5	162.00	989.00	170.00	989.00	3
6	170.00	989.00	232.00	1014.00	3
7	232.00	1014.00	240.00	1014.00	3
8	240.00	1014.00	275.00	1028.00	3
9	275.00	1028.00	304.00	1040.00	4
10	304.00	1040.00	312.00	1040.00	4
11	312.00	1040.00	375.00	1064.00	4
12	375.00	1064.00	395.00	1064.00	4
13	395.00	1064.00	458.00	1090.00	4
14	458.00	1090.00	466.00	1090.00	4
15	466.00	1090.00	529.00	1115.00	4
16	529.00	1115.00	537.00	1115.00	5
17	537.00	1115.00	600.00	1140.00	5
18	600.00	1140.00	608.00	1140.00	5
19	608.00	1140.00	685.00	1170.00	5
20	685.00	1170.00	700.00	1180.00	5
21	700.00	1180.00	740.00	1190.00	5
22	740.00	1190.00	750.00	1191.00	5
23	750.00	1191.00	770.00	1193.00	6
24	770.00	1193.00	810.00	1199.00	6
25	810.00	1199.00	820.00	1198.00	6
26	820.00	1198.00	830.00	1194.00	6
27	830.00	1194.00	840.00	1189.00	6
28	840.00	1189.00	900.00	1150.00	7
29	0.00	953.00	25.00	964.00	3
30	275.00	1026.00	275.01	800.00	3
31	529.00	1115.00	535.00	800.00	4
32	750.00	1191.00	750.01	800.00	5
33	840.00	1189.00	840.01	800.00	6

User Specified Y-Origin = 800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type	Total Unit Wt.	Saturated Unit Wt.	Cohesion Intercept	Friction Angle	Pore Pressure	Pressure Constant	Piez. Surface

No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)	No.
1	120.0	125.0	250.0	28.0	0.00	0.0	0
2	125.0	130.0	300.0	30.0	0.00	0.0	0
3	135.0	135.0	500.0	35.0	0.00	0.0	1
4	135.0	135.0	500.0	35.0	0.00	0.0	1
5	135.0	135.0	500.0	35.0	0.00	0.0	1
6	135.0	135.0	500.0	35.0	0.00	0.0	1
7	135.0	135.0	500.0	35.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS

5 soil type(s)

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	15.5	500.00	35.00
2	18.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 4 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.5	500.00	35.00
2	19.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 5 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	500.00	35.00
2	22.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 6 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	13.5	500.00	35.00
2	16.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 7 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	8.5	500.00	35.00
2	11.5	200.00	20.00
3	90.0	500.00	35.00

ANISOTROPIC SOIL NOTES:

(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.

(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.

(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	900.00
2	900.00	900.00

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been

Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 45.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	85.00	925.00	245.00	978.00	60.00
2	325.00	992.00	600.00	1090.00	85.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 48
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 1.6 %
Statistical Data On All Valid FS Values:
FS Max = 9.245 FS Min = 1.559 FS Ave = 3.226
Standard Deviation = 0.872 Coefficient of Variation = 27.04 %
((Simplified Janbu FS for Critical Surface = 1.521))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	103.98	965.60
2	136.87	961.47
3	466.11	1059.28
4	496.38	1092.58
5	507.41	1106.43

*** FOS = 1.559 Theta (ki=1.0) = 16.56 ***
Lambda = 0.297
Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	32.9	38619.7	0.0	0.0	0.	0.	0.0	0.0	0.0
2	25.1	63555.9	0.0	0.0	0.	0.	0.0	0.0	0.0
3	8.0	20386.4	0.0	0.0	0.	0.	0.0	0.0	0.0
4	62.0	175593.2	0.0	0.0	0.	0.	0.0	0.0	0.0
5	8.0	24928.0	0.0	0.0	0.	0.	0.0	0.0	0.0
6	35.0	111956.6	0.0	0.0	0.	0.	0.0	0.0	0.0
7	0.0	3.9	0.0	0.0	0.	0.	0.0	0.0	0.0
8	29.0	106438.3	0.0	0.0	0.	0.	0.0	0.0	0.0
9	8.0	29907.9	0.0	0.0	0.	0.	0.0	0.0	0.0
10	63.0	247891.9	0.0	0.0	0.	0.	0.0	0.0	0.0
11	20.0	77809.2	0.0	0.0	0.	0.	0.0	0.0	0.0
12	63.0	250811.3	0.0	0.0	0.	0.	0.0	0.0	0.0
13	8.0	34499.5	0.0	0.0	0.	0.	0.0	0.0	0.0
14	0.1	451.4	0.0	0.0	0.	0.	0.0	0.0	0.0
15	30.3	82226.7	0.0	0.0	0.	0.	0.0	0.0	0.0
16	11.0	7058.3	0.0	0.0	0.	0.	0.0	0.0	0.0

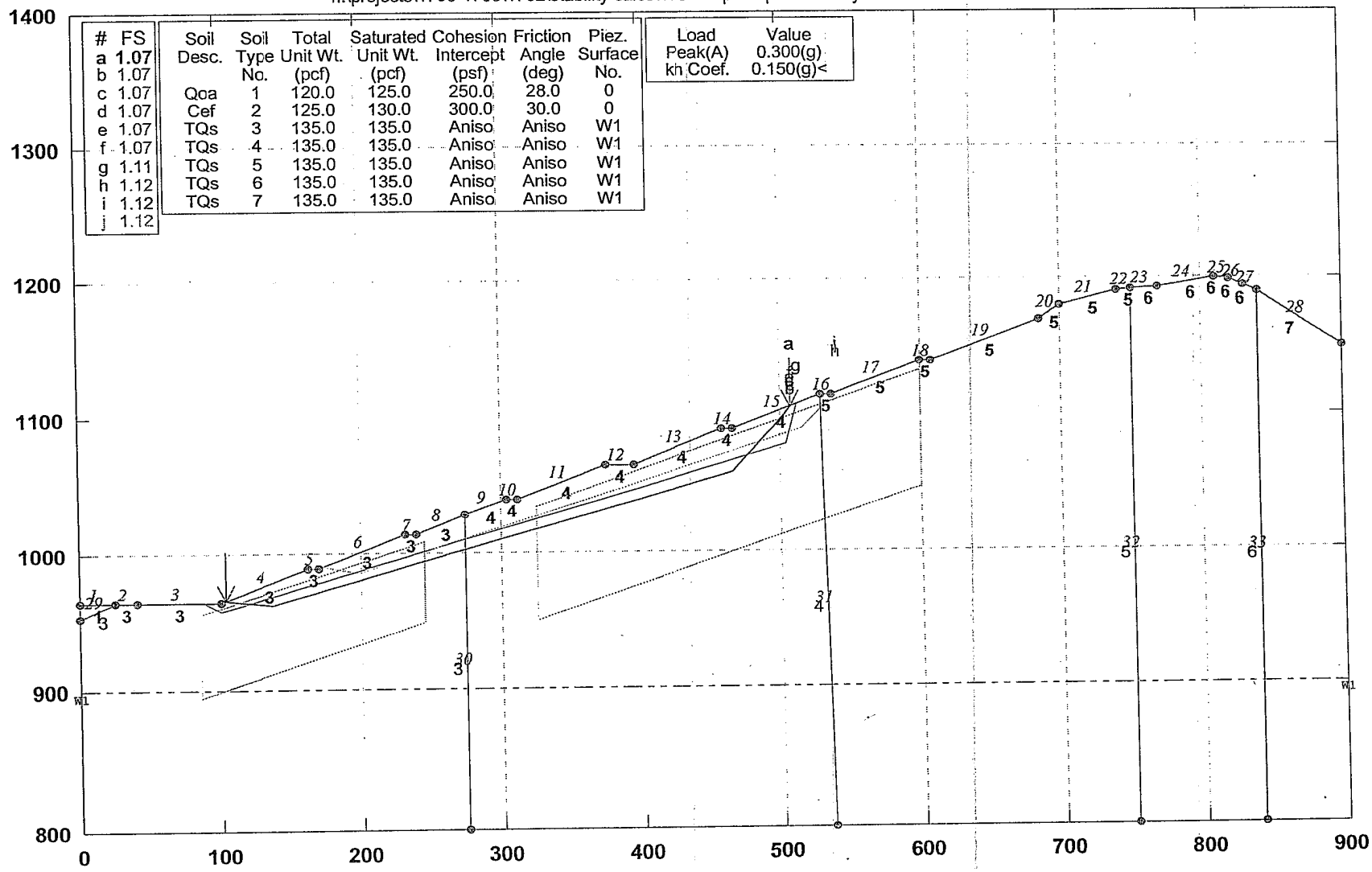
Lndmrk off-site grading; 5-5'; CS-4; pseudo; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t5ps2b.pl2 Run By: KPC 7/9/2008 10:30AM

#	FS
a	1.07
b	1.07
c	1.07
d	1.07
e	1.07
f	1.07
g	1.11
h	1.12
i	1.12
j	1.12

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qoa	1	120.0	125.0	250.0	28.0	0
Cef	2	125.0	130.0	300.0	30.0	0
TQs	3	135.0	135.0	Aniso	Aniso	W1
TQs	4	135.0	135.0	Aniso	Aniso	W1
TQs	5	135.0	135.0	Aniso	Aniso	W1
TQs	6	135.0	135.0	Aniso	Aniso	W1
TQs	7	135.0	135.0	Aniso	Aniso	W1

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.07

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 18

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 7/9/2008
Time of Run: 10:30AM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5ps2b.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5ps2b.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t5ps2b.PLT
PROBLEM DESCRIPTION: Lndmrk off-site grading; 5-5'; CS-4;
pseudo; planar

BOUNDARY COORDINATES
28 Top Boundaries
33 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	964.00	25.00	964.00	1
2	25.00	964.00	40.00	964.00	3
3	40.00	964.00	100.00	964.00	3
4	100.00	964.00	162.00	989.00	3
5	162.00	989.00	170.00	989.00	3
6	170.00	989.00	232.00	1014.00	3
7	232.00	1014.00	240.00	1014.00	3
8	240.00	1014.00	275.00	1028.00	3
9	275.00	1028.00	304.00	1040.00	4
10	304.00	1040.00	312.00	1040.00	4
11	312.00	1040.00	375.00	1064.00	4
12	375.00	1064.00	395.00	1064.00	4
13	395.00	1064.00	458.00	1090.00	4
14	458.00	1090.00	466.00	1090.00	4
15	466.00	1090.00	529.00	1115.00	4
16	529.00	1115.00	537.00	1115.00	5
17	537.00	1115.00	600.00	1140.00	5
18	600.00	1140.00	608.00	1140.00	5
19	608.00	1140.00	685.00	1170.00	5
20	685.00	1170.00	700.00	1180.00	5
21	700.00	1180.00	740.00	1190.00	5
22	740.00	1190.00	750.00	1191.00	5
23	750.00	1191.00	770.00	1193.00	6
24	770.00	1193.00	810.00	1199.00	6
25	810.00	1199.00	820.00	1198.00	6
26	820.00	1198.00	830.00	1194.00	6
27	830.00	1194.00	840.00	1189.00	6
28	840.00	1189.00	900.00	1150.00	7
29	0.00	953.00	25.00	964.00	3
30	275.00	1028.00	275.01	800.00	3
31	529.00	1115.00	535.00	800.00	4
32	750.00	1191.00	750.01	800.00	5
33	840.00	1189.00	840.01	800.00	6

User Specified Y-Origin = 800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type	Total Unit Wt.	Saturated Unit Wt.	Cohesion Intercept	Friction Angle	Pore Pressure	Pressure Constant	Piez. Surface

No.	(pcf)	(pcf)	(psf)	(deg)	Param.	(psf)	No.
1	120.0	125.0	250.0	28.0	0.00	0.0	0
2	125.0	130.0	300.0	30.0	0.00	0.0	0
3	135.0	135.0	600.0	38.0	0.00	0.0	1
4	135.0	135.0	600.0	38.0	0.00	0.0	1
5	135.0	135.0	600.0	38.0	0.00	0.0	1
6	135.0	135.0	600.0	38.0	0.00	0.0	1
7	135.0	135.0	600.0	38.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS

5 soil type(s)

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	15.5	600.00	38.00
2	18.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 4 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.5	600.00	38.00
2	19.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 5 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	600.00	38.00
2	22.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 6 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	13.5	600.00	38.00
2	16.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 7 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	8.5	600.00	38.00
2	11.5	200.00	20.00
3	90.0	600.00	38.00

ANISOTROPIC SOIL NOTES:

(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.

(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.

(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	900.00
2	900.00	900.00

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 45.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	85.00	925.00	245.00	978.00	60.00
2	325.00	992.00	600.00	1090.00	85.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 300
Number of Trial Surfaces with Misleading FS = 5
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 10.2 %
Statistical Data On All Valid FS Values:
FS Max = 5.443 FS Min = 1.067 FS Ave = 2.380
Standard Deviation = 0.542 Coefficient of Variation = 22.78 %
((Simplified Janbu FS for Critical Surface = 1.035))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	103.98	965.60
2	136.87	961.47
3	466.11	1059.23
4	496.38	1092.59
5	507.41	1106.43

*** FOS = 1.067 Theta (ki=1.0) = 14.63 ***
Lambda = 0.261
Individual data on the 16 slices

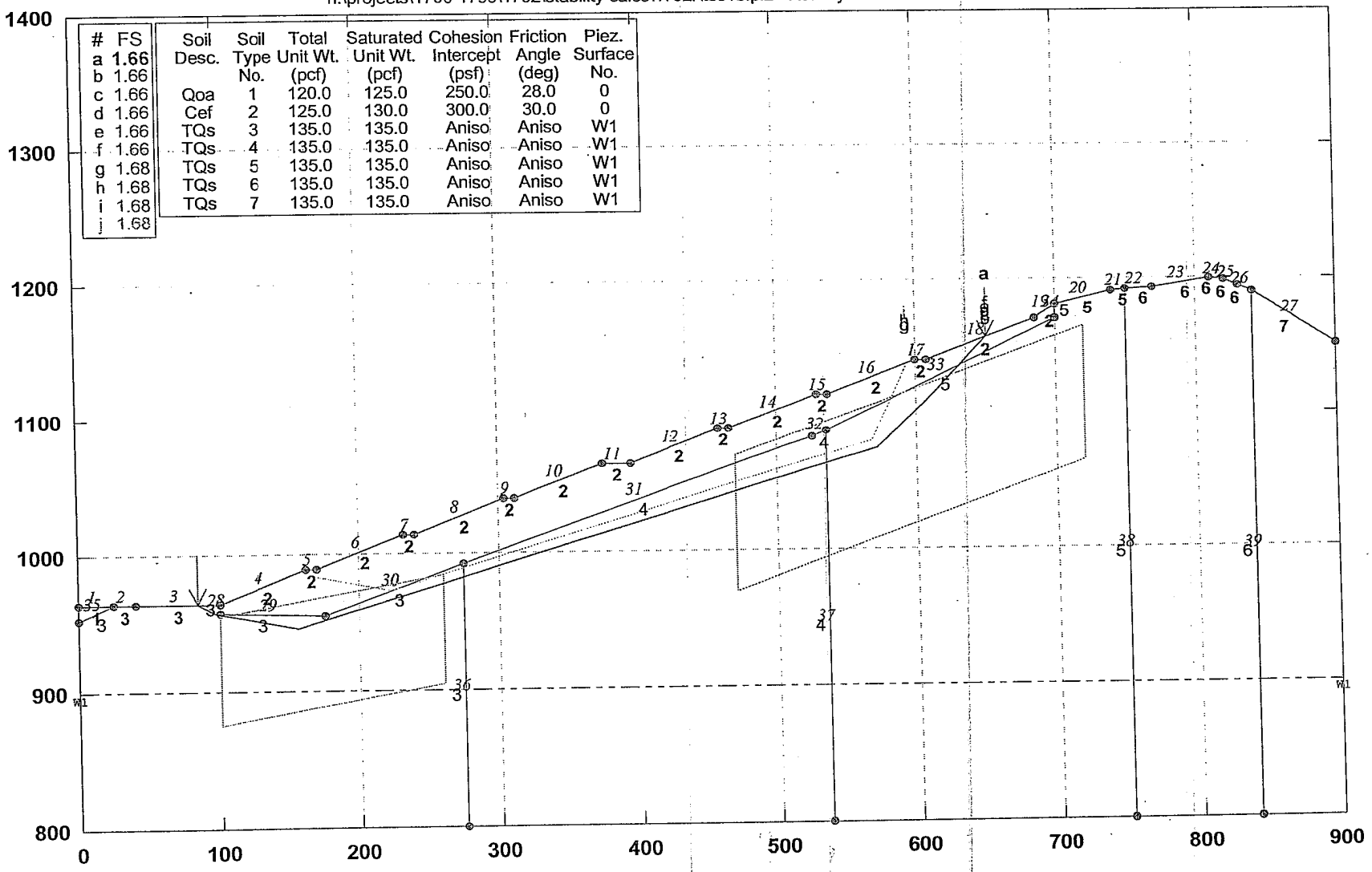
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	32.9	38619.7	0.0	0.0	0.	0.	5792.9	0.0	0.0
2	25.1	63555.9	0.0	0.0	0.	0.	9533.4	0.0	0.0
3	8.0	20386.4	0.0	0.0	0.	0.	3058.0	0.0	0.0
4	62.0	175593.2	0.0	0.0	0.	0.	26339.0	0.0	0.0
5	8.0	24928.0	0.0	0.0	0.	0.	3739.2	0.0	0.0
6	35.0	111956.6	0.0	0.0	0.	0.	16793.5	0.0	0.0
7	0.0	3.9	0.0	0.0	0.	0.	0.6	0.0	0.0
8	29.0	106438.3	0.0	0.0	0.	0.	15965.7	0.0	0.0
9	8.0	29907.9	0.0	0.0	0.	0.	4486.2	0.0	0.0
10	63.0	247891.9	0.0	0.0	0.	0.	37183.8	0.0	0.0
11	20.0	77809.2	0.0	0.0	0.	0.	11671.4	0.0	0.0
12	63.0	250811.3	0.0	0.0	0.	0.	37621.7	0.0	0.0
13	8.0	34499.5	0.0	0.0	0.	0.	5174.9	0.0	0.0
14	0.1	451.4	0.0	0.0	0.	0.	67.7	0.0	0.0
15	30.3	82226.7	0.0	0.0	0.	0.	12334.0	0.0	0.0
16	11.0	7058.3	0.0	0.0	0.	0.	1058.8	0.0	0.0

Lndmrk off-site grading; 5-5'; CS-4 w/ 7x75 buttress; static; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t5s1b.pl2 Run By: KPC 7/10/2008 11:36AM

#	FS
a	1.66
b	1.66
c	1.66
d	1.66
e	1.66
f	1.66
g	1.68
h	1.68
i	1.68
j	1.68

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qoa	1	120.0	125.0	250.0	28.0	0
Cef	2	125.0	130.0	300.0	30.0	0
TQs	3	135.0	135.0	Aniso	Aniso	W1
TQs	4	135.0	135.0	Aniso	Aniso	W1
TQs	5	135.0	135.0	Aniso	Aniso	W1
TQs	6	135.0	135.0	Aniso	Aniso	W1
TQs	7	135.0	135.0	Aniso	Aniso	W1



GSTABL7 v.2 FSmin=1.66

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 19

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 7/10/2008
Time of Run: 11:36AM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5s1b.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5s1b.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t5s1b.PLT
PROBLEM DESCRIPTION: Lndmrk off-site grading; 5-5'; CS-4
w/ 7x75 buttress; static; planar

BOUNDARY COORDINATES
27 Top Boundaries
39 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	964.00	25.00	964.00	1
2	25.00	964.00	40.00	964.00	3
3	40.00	964.00	100.00	964.00	3
4	100.00	964.00	162.00	989.00	2
5	162.00	989.00	170.00	989.00	2
6	170.00	989.00	232.00	1014.00	2
7	232.00	1014.00	240.00	1014.00	2
8	240.00	1014.00	304.00	1040.00	2
9	304.00	1040.00	312.00	1040.00	2
10	312.00	1040.00	375.00	1064.00	2
11	375.00	1064.00	395.00	1064.00	2
12	395.00	1064.00	458.00	1090.00	2
13	458.00	1090.00	466.00	1090.00	2
14	466.00	1090.00	529.00	1115.00	2
15	529.00	1115.00	537.00	1115.00	2
16	537.00	1115.00	600.00	1140.00	2
17	600.00	1140.00	608.00	1140.00	2
18	608.00	1140.00	685.00	1170.00	2
19	685.00	1170.00	700.00	1180.00	2
20	700.00	1180.00	740.00	1190.00	5
21	740.00	1190.00	750.00	1191.00	5
22	750.00	1191.00	770.00	1193.00	6
23	770.00	1193.00	810.00	1199.00	6
24	810.00	1199.00	820.00	1198.00	6
25	820.00	1190.00	830.00	1194.00	6
26	830.00	1194.00	840.00	1189.00	6
27	840.00	1189.00	900.00	1150.00	7
28	100.00	964.00	100.01	957.00	3
29	100.01	957.00	175.00	955.50	3
30	175.00	955.50	275.00	992.00	3
31	275.00	992.00	525.00	1083.00	4
32	525.00	1083.00	535.00	1088.00	4
33	535.00	1088.00	699.99	1170.00	5
34	699.99	1170.00	700.00	1180.00	5
35	0.00	953.00	25.00	964.00	3
36	275.00	992.00	275.01	800.00	3
37	535.00	1088.00	535.01	800.00	4
38	750.00	1191.00	750.01	800.00	5
39	840.00	1189.00	840.01	800.00	6

User Specified Y-Origin = 800.00(ft)

Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	125.0	250.0	28.0	0.00	0.0	0
2	125.0	130.0	300.0	30.0	0.00	0.0	0
3	135.0	135.0	500.0	35.0	0.00	0.0	1
4	135.0	135.0	500.0	35.0	0.00	0.0	1
5	135.0	135.0	500.0	35.0	0.00	0.0	1
6	135.0	135.0	500.0	35.0	0.00	0.0	1
7	135.0	135.0	500.0	35.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS

5 soil type(s)

Soil Type 3 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	15.5	500.00	35.00
2	18.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 4 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.5	500.00	35.00
2	19.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 5 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	500.00	35.00
2	22.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 6 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	13.5	500.00	35.00
2	16.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 7 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	8.5	500.00	35.00
2	11.5	200.00	20.00
3	90.0	500.00	35.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	900.00
2	900.00	900.00

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 60.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	100.00	915.00	260.00	945.00	80.00
2	470.00	1020.00	720.00	1114.00	100.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0=2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 14
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.5 %
Statistical Data On All Valid FS Values:
FS Max = 6.727 FS Min = 1.663 FS Ave = 3.089
Standard Deviation = 0.747 Coefficient of Variation = 24.19 %
(Simplified Janbu FS for Critical Surface = 1.575))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	84.21	964.00
2	97.25	957.93
3	156.08	946.16
4	571.67	1074.21
5	613.93	1116.80
6	649.73	1156.26

*** FOS = 1.663 Theta (ki=1.0) = 16.11 ***
Lambda = 0.289
Individual data on the 27 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	13.0	5342.7	0.0	0.0	0.	0.	0.0	0.0	0.0
2	2.7	2355.6	0.0	0.0	0.	0.	0.0	0.0	0.0
3	0.0	8.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	2.1	1898.1	0.0	0.0	0.	0.	0.0	0.0	0.0
5	54.0	165746.9	0.0	0.0	0.	0.	0.0	0.0	0.0
6	5.9	30645.4	0.0	0.0	0.	0.	0.0	0.0	0.0
7	8.0	40306.7	0.0	0.0	0.	0.	0.0	0.0	0.0
8	5.0	24463.2	0.0	0.0	0.	0.	0.0	0.0	0.0
9	57.0	300339.6	0.0	0.0	0.	0.	0.0	0.0	0.0
10	8.0	43778.2	0.0	0.0	0.	0.	0.0	0.0	0.0
11	35.0	194080.1	0.0	0.0	0.	0.	0.0	0.0	0.0
12	29.0	172708.5	0.0	0.0	0.	0.	0.0	0.0	0.0
13	8.0	47917.3	0.0	0.0	0.	0.	0.0	0.0	0.0
14	63.0	386954.1	0.0	0.0	0.	0.	0.0	0.0	0.0

15	20.0	121336.9	0.0	0.0	0.	0.	0.0	0.0	0.0
16	63.0	385343.5	0.0	0.0	0.	0.	0.0	0.0	0.0
17	8.0	51152.3	0.0	0.0	0.	0.	0.0	0.0	0.0
18	59.0	388557.0	0.0	0.0	0.	0.	0.0	0.0	0.0
19	4.0	27821.0	0.0	0.0	0.	0.	0.0	0.0	0.0
20	6.0	41234.6	0.0	0.0	0.	0.	0.0	0.0	0.0
21	2.0	13444.3	0.0	0.0	0.	0.	0.0	0.0	0.0
22	34.7	239679.8	0.0	0.0	0.	0.	0.0	0.0	0.0
23	28.3	169538.7	0.0	0.0	0.	0.	0.0	0.0	0.0
24	8.0	34448.5	0.0	0.0	0.	0.	0.0	0.0	0.0
25	5.9	20988.6	0.0	0.0	0.	0.	0.0	0.0	0.0
26	17.2	42622.1	0.0	0.0	0.	0.	0.0	0.0	0.0
27	18.6	15354.7	0.0	0.0	0.	0.	0.0	0.0	0.0

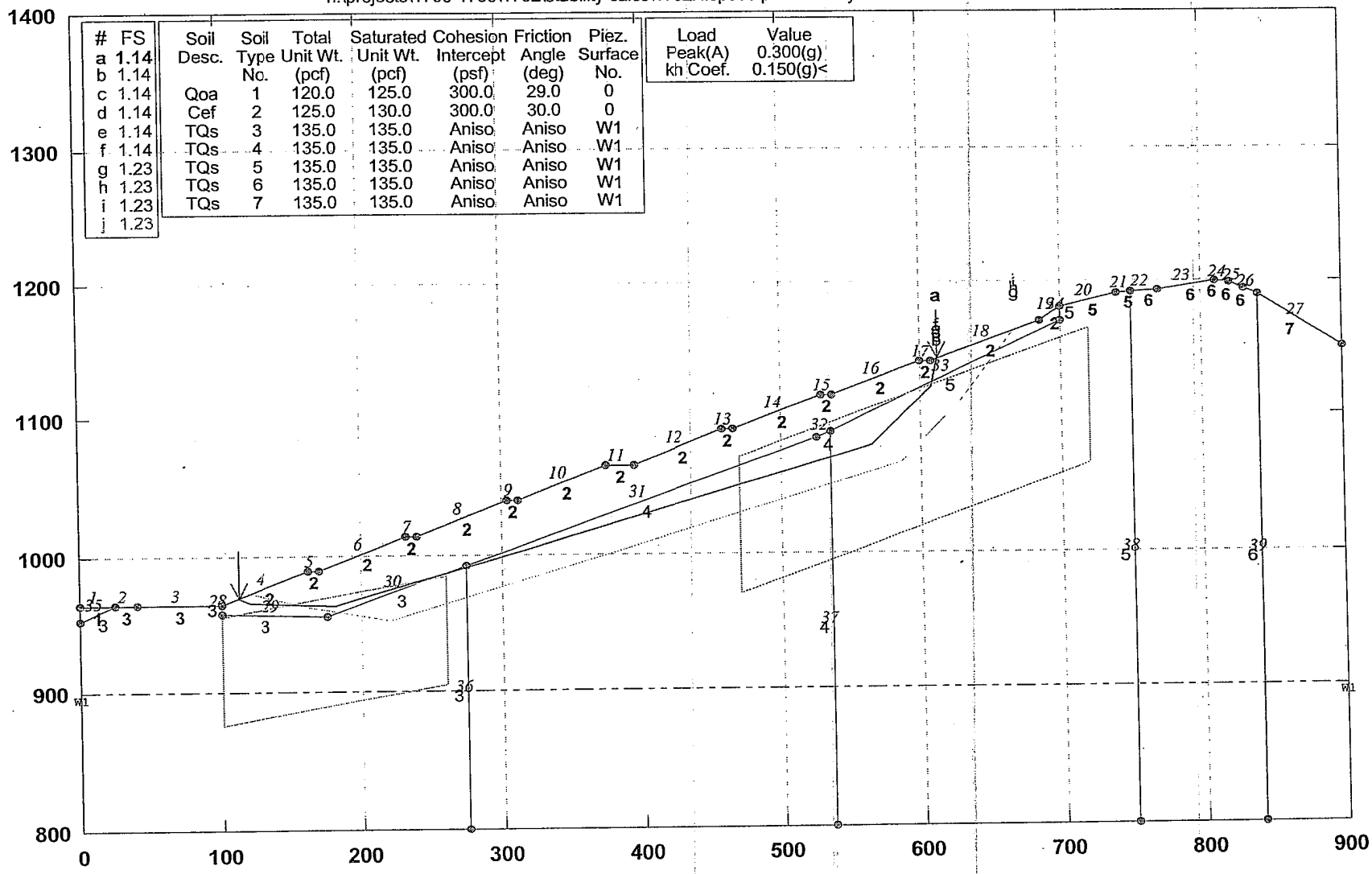
Lndmrk off-site grading; 5-5'; CS-4 w/ 7x75 buttress; pseduo; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t5ps1b.pl2 Run By: KPC 7/10/2008 01:13PM

#	FS
a	1.14
b	1.14
c	1.14
d	1.14
e	1.14
f	1.14
g	1.23
h	1.23
i	1.23
j	1.23

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qoa	1	120.0	125.0	300.0	29.0	0
Cef	2	125.0	130.0	300.0	30.0	0
TQs	3	135.0	135.0	Aniso	Aniso	W1
TQs	4	135.0	135.0	Aniso	Aniso	W1
TQs	5	135.0	135.0	Aniso	Aniso	W1
TQs	6	135.0	135.0	Aniso	Aniso	W1
TQs	7	135.0	135.0	Aniso	Aniso	W1

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.14

Safety Factors Are Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 20

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                              *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
          ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                        (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                        SLOPE STABILITY ANALYSIS SYSTEM
             Modified Bishop, Simplified Janbu, or GLE Method of Slices.
             (Includes Spencer & Morgenstern-Price Type Analysis)
             Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
             Nonlinear Undrained Shear Strength, Curved Phi Envelope,
             Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
             Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/10/2008
Time of Run:                01:13PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5ps1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t5ps1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t5ps1b.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 5-5'; CS-4
                      w/ 7x75 buttress; pseduo; planar

BOUNDARY COORDINATES
   27 Top   Boundaries
   39 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	964.00	25.00	964.00	1
2	25.00	964.00	40.00	964.00	3
3	40.00	964.00	100.00	964.00	3
4	100.00	964.00	162.00	989.00	2
5	162.00	989.00	170.00	989.00	2
6	170.00	989.00	232.00	1014.00	2
7	232.00	1014.00	240.00	1014.00	2
8	240.00	1014.00	304.00	1040.00	2
9	304.00	1040.00	312.00	1040.00	2
10	312.00	1040.00	375.00	1064.00	2
11	375.00	1064.00	395.00	1064.00	2
12	395.00	1064.00	458.00	1090.00	2
13	458.00	1090.00	466.00	1090.00	2
14	466.00	1090.00	529.00	1115.00	2
15	529.00	1115.00	537.00	1115.00	2
16	537.00	1115.00	600.00	1140.00	2
17	600.00	1140.00	608.00	1140.00	2
18	608.00	1140.00	685.00	1170.00	2
19	685.00	1170.00	700.00	1180.00	2
20	700.00	1180.00	740.00	1190.00	5
21	740.00	1190.00	750.00	1191.00	5
22	750.00	1191.00	770.00	1193.00	6
23	770.00	1193.00	810.00	1199.00	6
24	810.00	1199.00	820.00	1198.00	6
25	820.00	1198.00	830.00	1194.00	6
26	830.00	1194.00	840.00	1189.00	6
27	840.00	1189.00	900.00	1150.00	7
28	100.00	964.00	100.01	957.00	3
29	100.01	957.00	175.00	955.50	3
30	175.00	955.50	275.00	992.00	3
31	275.00	992.00	525.00	1083.00	4
32	525.00	1083.00	535.00	1088.00	4
33	535.00	1088.00	699.99	1170.00	5
34	699.99	1170.00	700.00	1180.00	5
35	0.00	953.00	25.00	964.00	3
36	275.00	992.00	275.01	800.00	3
37	535.00	1088.00	535.01	800.00	4
38	750.00	1191.00	750.01	800.00	5
39	840.00	1189.00	840.01	800.00	6

User Specified Y-Origin = 800.00(ft)

Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
7 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	125.0	300.0	29.0	0.00	0.0	0
2	125.0	130.0	300.0	30.0	0.00	0.0	0
3	135.0	135.0	600.0	38.0	0.00	0.0	1
4	135.0	135.0	600.0	38.0	0.00	0.0	1
5	135.0	135.0	600.0	38.0	0.00	0.0	1
6	135.0	135.0	600.0	38.0	0.00	0.0	1
7	135.0	135.0	600.0	38.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
5 soil type(s)

Soil Type 3 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	15.5	600.00	38.00
2	18.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 4 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.5	600.00	38.00
2	19.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 5 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	600.00	38.00
2	22.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 6 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	13.5	600.00	38.00
2	16.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 7 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	8.5	600.00	38.00
2	11.5	200.00	20.00
3	90.0	600.00	38.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	900.00
2	900.00	900.00

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 60.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	100.00	915.00	260.00	945.00	80.00
2	470.00	1020.00	720.00	1114.00	100.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 403
Number of Trial Surfaces with Misleading FS = 39
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 14.7 %
Statistical Data On All Valid FS Values:
FS Max = 4.829 FS Min = 1.141 FS Ave = 2.275
Standard Deviation = 0.500 Coefficient of Variation = 21.99 %
((Simplified Janbu FS for Critical Surface = 1.074))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	112.12	968.89
2	120.67	965.34
3	180.62	962.71
4	565.08	1078.08
5	607.01	1121.00
6	612.22	1141.64

*** FOS = 1.141 Theta (ki=1.0) = 15.85 ***
Lambda = 0.284
Individual data on the 24 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	8.6	3738.3	0.0	0.0	0.	0.	560.7	0.0	0.0
2	41.3	83854.6	0.0	0.0	0.	0.	12578.2	0.0	0.0
3	8.0	25649.2	0.0	0.0	0.	0.	3847.4	0.0	0.0
4	10.6	37416.9	0.0	0.0	0.	0.	5612.5	0.0	0.0
5	51.4	213388.0	0.0	0.0	0.	0.	32008.2	0.0	0.0
6	8.0	34671.4	0.0	0.0	0.	0.	5200.7	0.0	0.0
7	20.1	86729.5	0.0	0.0	0.	0.	13009.4	0.0	0.0
8	14.9	67906.4	0.0	0.0	0.	0.	10186.0	0.0	0.0
9	29.0	140930.5	0.0	0.0	0.	0.	21139.6	0.0	0.0

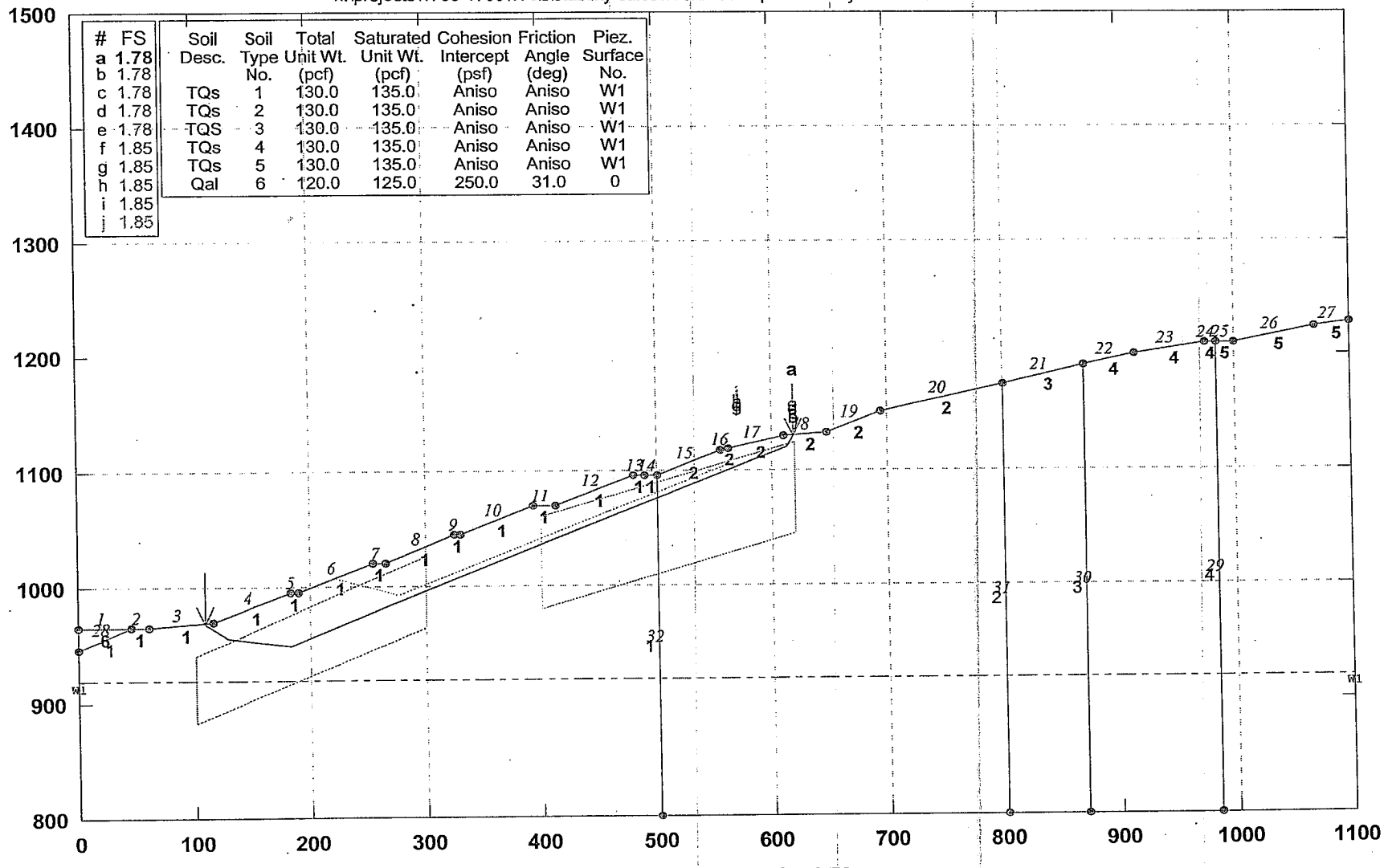
10	8.0	39311.0	0.0	0.0	0.	0.	5896.6	0.0	0.0
11	63.0	321608.8	0.0	0.0	0.	0.	48241.3	0.0	0.0
12	20.0	101494.0	0.0	0.0	0.	0.	15224.1	0.0	0.0
13	63.0	325678.8	0.0	0.0	0.	0.	48851.8	0.0	0.0
14	8.0	43884.2	0.0	0.0	0.	0.	6582.6	0.0	0.0
15	59.0	337102.4	0.0	0.0	0.	0.	50565.4	0.0	0.0
16	4.0	24469.4	0.0	0.0	0.	0.	3670.4	0.0	0.0
17	6.0	36237.9	0.0	0.0	0.	0.	5435.7	0.0	0.0
18	2.0	11789.8	0.0	0.0	0.	0.	1768.5	0.0	0.0
19	28.1	170114.9	0.0	0.0	0.	0.	25517.2	0.0	0.0
20	34.9	167504.9	0.0	0.0	0.	0.	25125.7	0.0	0.0
21	7.0	20116.0	0.0	0.0	0.	0.	3017.4	0.0	0.0
22	0.8	1763.3	0.0	0.0	0.	0.	264.5	0.0	0.0
23	0.2	358.5	0.0	0.0	0.	0.	53.8	0.0	0.0
24	4.2	3975.0	0.0	0.0	0.	0.	596.3	0.0	0.0

Lndmrk off-site grading; 6-6'; CS-4; static; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t6s1b.pl2 Run By: KPC 7/09/2008 01:20PM

#	FS
a	1.78
b	1.78
c	1.78
d	1.78
e	1.78
f	1.85
g	1.85
h	1.85
i	1.85
j	1.85

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
TQs	1	130.0	135.0	Aniso	Aniso	W1
TQs	2	130.0	135.0	Aniso	Aniso	W1
TQS	3	130.0	135.0	Aniso	Aniso	W1
TQs	4	130.0	135.0	Aniso	Aniso	W1
TQs	5	130.0	135.0	Aniso	Aniso	W1
Qal	6	120.0	125.0	250.0	31.0	0



GSTABL7 v.2 FSmin=1.78

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 21

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/09/2008
Time of Run:                01:20PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t6s1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t6s1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t6s1b.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 6-6'; CS-4;
                      static; planar

BOUNDARY COORDINATES
   27 Top   Boundaries
   32 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	967.00	45.00	967.00	6
2	45.00	967.00	60.00	967.00	1
3	60.00	967.00	115.00	970.00	1
4	115.00	970.00	183.00	995.00	1
5	183.00	995.00	190.00	995.00	1
6	190.00	995.00	255.00	1020.00	1
7	255.00	1020.00	265.00	1020.00	1
8	265.00	1020.00	325.00	1045.00	1
9	325.00	1045.00	330.00	1045.00	1
10	330.00	1045.00	393.00	1069.00	1
11	393.00	1069.00	413.00	1070.00	1
12	413.00	1070.00	480.00	1095.00	1
13	480.00	1095.00	490.00	1096.00	1
14	490.00	1096.00	500.00	1096.00	1
15	500.00	1096.00	555.00	1118.00	2
16	555.00	1118.00	562.00	1119.00	2
17	562.00	1119.00	610.00	1130.00	2
18	610.00	1130.00	647.00	1133.00	2
19	647.00	1133.00	695.00	1150.00	2
20	695.00	1150.00	800.00	1174.00	2
21	800.00	1174.00	870.00	1190.00	3
22	870.00	1190.00	915.00	1200.00	4
23	915.00	1200.00	975.00	1210.00	4
24	975.00	1210.00	985.00	1210.00	4
25	985.00	1210.00	1000.00	1210.00	5
26	1000.00	1210.00	1070.00	1223.00	5
27	1070.00	1223.00	1100.00	1228.00	5
28	0.00	947.00	45.00	967.00	1
29	985.00	1210.00	985.01	800.00	4
30	870.00	1190.00	870.01	800.00	3
31	800.00	1174.00	800.01	800.00	2
32	500.00	1096.00	500.01	800.00	1

```
User Specified Y-Origin =        800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  6 Type(s) of Soil
```

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.

1	130.0	135.0	500.0	35.0	0.00	0.0	1
2	130.0	135.0	500.0	35.0	0.00	0.0	1
3	130.0	135.0	500.0	35.0	0.00	0.0	1
4	130.0	135.0	500.0	35.0	0.00	0.0	1
5	130.0	135.0	500.0	35.0	0.00	0.0	1
6	120.0	125.0	250.0	31.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS
5 soil type(s)

Soil Type 1 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	21.5	500.00	35.00
2	24.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 2 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	500.00	35.00
2	22.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 3 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	14.5	500.00	35.00
2	17.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 4 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	10.5	500.00	35.00
2	13.5	200.00	20.00
3	90.0	500.00	35.00

Soil Type 5 Is Anisotropic
Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	5.5	500.00	35.00
2	8.5	200.00	20.00
3	90.0	500.00	35.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED
Unit Weight of Water = 62.40 (pcf)
Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	920.00
2	1100.00	920.00

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
2500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 55.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	100.00	912.00	300.00	955.00	60.00
2	400.00	1020.00	620.00	1085.00	80.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 17
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.7 %
Statistical Data On All Valid FS Values:
FS Max = 5.601 FS Min = 1.778 FS Ave = 3.383
Standard Deviation = 0.610 Coefficient of Variation = 18.04 %
((Simplified Janbu FS for Critical Surface = 1.576))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	108.91	969.67
2	127.93	956.52
3	182.60	950.51
4	613.22	1120.78
5	618.26	1130.67

*** FOS = 1.778 Theta (ki=1.0) = 16.49 ***
Lambda = 0.296
Individual data on the 19 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	6.1	1797.3	0.0	0.0	0.	0.	0.0	0.0	0.0
2	12.9	19147.2	0.0	0.0	0.	0.	0.0	0.0	0.0
3	54.7	222376.0	0.0	0.0	0.	0.	0.0	0.0	0.0
4	0.4	2276.9	0.0	0.0	0.	0.	0.0	0.0	0.0
5	7.0	39083.6	0.0	0.0	0.	0.	0.0	0.0	0.0
6	65.0	348257.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	10.0	50551.2	0.0	0.0	0.	0.	0.0	0.0	0.0
8	60.0	292857.5	0.0	0.0	0.	0.	0.0	0.0	0.0
9	5.0	24176.5	0.0	0.0	0.	0.	0.0	0.0	0.0
10	63.0	292795.0	0.0	0.0	0.	0.	0.0	0.0	0.0
11	20.0	82784.9	0.0	0.0	0.	0.	0.0	0.0	0.0
12	67.0	240740.3	0.0	0.0	0.	0.	0.0	0.0	0.0
13	10.0	33040.5	0.0	0.0	0.	0.	0.0	0.0	0.0
14	10.0	28551.8	0.0	0.0	0.	0.	0.0	0.0	0.0
15	55.0	143787.3	0.0	0.0	0.	0.	0.0	0.0	0.0
16	7.0	17610.7	0.0	0.0	0.	0.	0.0	0.0	0.0
17	48.0	90344.4	0.0	0.0	0.	0.	0.0	0.0	0.0
18	3.2	4177.3	0.0	0.0	0.	0.	0.0	0.0	0.0
19	5.0	3103.8	0.0	0.0	0.	0.	0.0	0.0	0.0

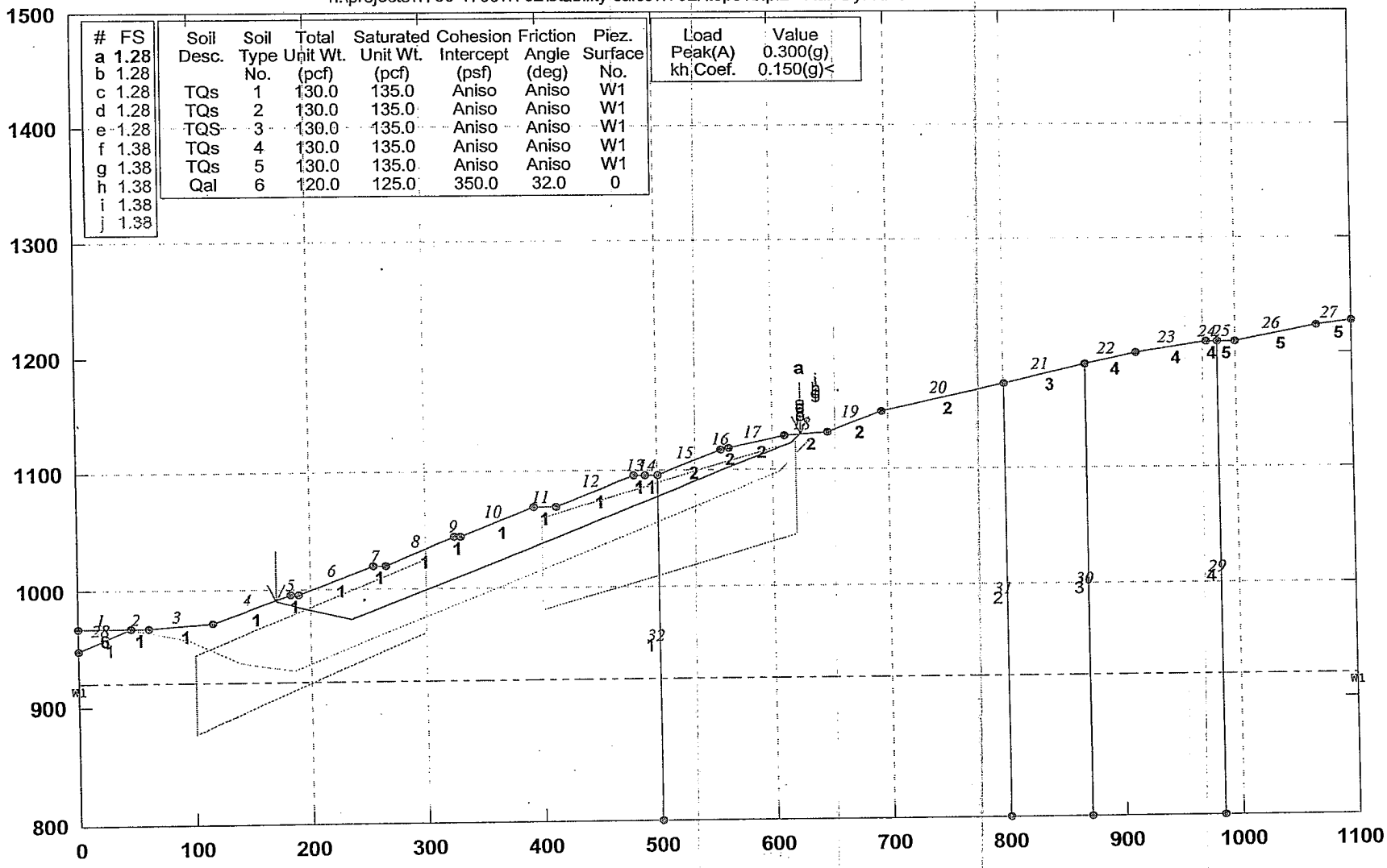
Lndmrk off-site grading; 6-6'; CS-4; pseudo; planar

n:\projects\1700-1799\1702\stability calcs\1702r\t6ps1b.pl2 Run By: KPC 7/09/2008 01:24PM

#	FS
a	1.28
b	1.28
c	1.28
d	1.28
e	1.28
f	1.38
g	1.38
h	1.38
i	1.38
j	1.38

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
TQs	1	130.0	135.0	Aniso	Aniso	W1
TQs	2	130.0	135.0	Aniso	Aniso	W1
TQS	3	130.0	135.0	Aniso	Aniso	W1
TQs	4	130.0	135.0	Aniso	Aniso	W1
TQs	5	130.0	135.0	Aniso	Aniso	W1
Qal	6	120.0	125.0	350.0	32.0	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.28

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 22

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/09/2003
Time of Run:                01:24PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t6ps1b.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t6ps1b.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t6ps1b.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 6-6'; CS-4;
                      pseudo; planar
BOUNDARY COORDINATES
   27 Top   Boundaries
   32 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	967.00	45.00	967.00	6
2	45.00	967.00	60.00	967.00	1
3	60.00	967.00	115.00	970.00	1
4	115.00	970.00	183.00	995.00	1
5	183.00	995.00	190.00	995.00	1
6	190.00	995.00	255.00	1020.00	1
7	255.00	1020.00	265.00	1020.00	-
8	265.00	1020.00	325.00	1045.00	1
9	325.00	1045.00	330.00	1045.00	1
10	330.00	1045.00	393.00	1069.00	1
11	393.00	1069.00	413.00	1070.00	1
12	413.00	1070.00	480.00	1095.00	1
13	480.00	1095.00	490.00	1096.00	1
14	490.00	1096.00	500.00	1096.00	1
15	500.00	1096.00	555.00	1118.00	2
16	555.00	1118.00	562.00	1119.00	2
17	562.00	1119.00	610.00	1130.00	2
18	610.00	1130.00	647.00	1133.00	2
19	647.00	1133.00	695.00	1150.00	2
20	695.00	1150.00	800.00	1174.00	2
21	800.00	1174.00	870.00	1190.00	3
22	870.00	1190.00	915.00	1200.00	4
23	915.00	1200.00	975.00	1210.00	4
24	975.00	1210.00	985.00	1210.00	4
25	985.00	1210.00	1000.00	1210.00	5
26	1000.00	1210.00	1070.00	1223.00	5
27	1070.00	1223.00	1100.00	1228.00	5
28	0.00	947.00	45.00	967.00	1
29	985.00	1210.00	985.01	800.00	4
30	870.00	1190.00	870.01	800.00	3
31	800.00	1174.00	800.01	800.00	2
32	500.00	1096.00	500.01	800.00	1

```
 User Specified Y-Origin =        800.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  6 Type(s) of Soil
```

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parem.	Pressure Constant (psf)	Piez. Surface No.

1	130.0	135.0	600.0	38.0	0.00	0.0	1
2	130.0	135.0	600.0	38.0	0.00	0.0	1
3	130.0	135.0	600.0	38.0	0.00	0.0	1
4	130.0	135.0	600.0	38.0	0.00	0.0	1
5	130.0	135.0	600.0	38.0	0.00	0.0	1
6	120.0	125.0	350.0	32.0	0.00	0.0	0

ANISOTROPIC STRENGTH PARAMETERS

5 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	21.5	600.00	38.00
2	24.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	19.5	600.00	38.00
2	22.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 3 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	14.5	600.00	38.00
2	17.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 4 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	10.5	600.00	38.00
2	13.5	200.00	20.00
3	90.0	600.00	38.00

Soil Type 5 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	5.5	600.00	38.00
2	8.5	200.00	20.00
3	90.0	600.00	38.00

ANISOTROPIC SOIL NOTES:

(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.

(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.

(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points

Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	920.00
2	1100.00	920.00

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)

Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
2500 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 47.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	100.00	910.00	390.00	993.00	65.00
2	400.00	1020.00	620.00	1085.00	80.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 214
Number of Trial Surfaces with Misleading FS = 21
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 9.4 %
Statistical Data On All Valid FS Values:
FS Max = 5.626 FS Min = 1.279 FS Ave = 2.468
Standard Deviation = 0.403 Coefficient of Variation = 16.32 %
((Simplified Janbu FS for Critical Surface = 1.123))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	169.93	990.19
2	190.29	984.48
3	235.78	972.66
4	615.42	1122.47
5	623.64	1131.11

*** FOS = 1.279 Theta (ki=1.0) = 15.95 ***
Lambda = 0.286
Individual data on the 18 slices

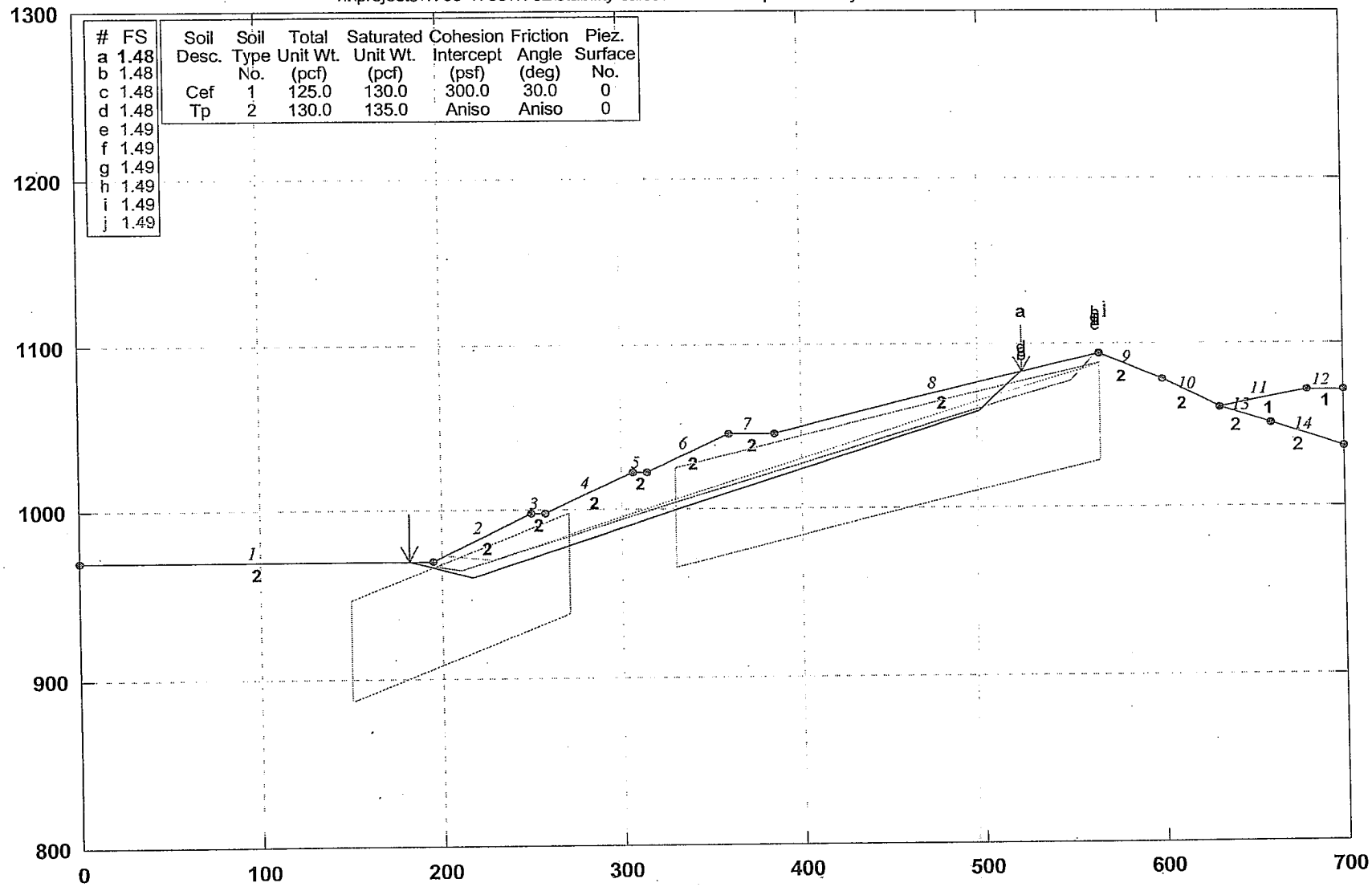
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	13.1	7201.2	0.0	0.0	0.	0.	1080.2	0.0	0.0
2	7.0	8608.1	0.0	0.0	0.	0.	1291.2	0.0	0.0
3	0.3	395.1	0.0	0.0	0.	0.	59.3	0.0	0.0
4	45.5	149563.3	0.0	0.0	0.	0.	22434.5	0.0	0.0
5	19.2	99577.9	0.0	0.0	0.	0.	14936.7	0.0	0.0
6	10.0	49116.5	0.0	0.0	0.	0.	7367.5	0.0	0.0
7	60.0	284471.0	0.0	0.0	0.	0.	42670.7	0.0	0.0
8	5.0	23494.8	0.0	0.0	0.	0.	3524.2	0.0	0.0
9	63.0	284431.5	0.0	0.0	0.	0.	42664.7	0.0	0.0
10	20.0	80217.6	0.0	0.0	0.	0.	12032.6	0.0	0.0
11	67.0	232447.1	0.0	0.0	0.	0.	34867.1	0.0	0.0
12	10.0	31843.3	0.0	0.0	0.	0.	4776.5	0.0	0.0
13	10.0	27365.1	0.0	0.0	0.	0.	4104.8	0.0	0.0
14	55.0	137450.0	0.0	0.0	0.	0.	20617.5	0.0	0.0
15	7.0	16826.9	0.0	0.0	0.	0.	2524.0	0.0	0.0
16	48.0	85109.9	0.0	0.0	0.	0.	12766.5	0.0	0.0
17	5.4	6215.1	0.0	0.0	0.	0.	932.3	0.0	0.0
18	8.2	4257.1	0.0	0.0	0.	0.	638.6	0.0	0.0

Lndmrk off-site grading; 7-7'; CS-3; planar; static

n:\projects\1700-1799\1702\stability calcs\1702r\t7s1a.pl2 Run By: KPC 7/9/2008 03:56PM

#	FS
a	1.48
b	1.48
c	1.48
d	1.48
e	1.49
f	1.49
g	1.49
h	1.49
i	1.49
j	1.49

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0
Tp	2	130.0	135.0	Aniso	Aniso	0



GSTABL7 v.2 FSmin=1.48

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 23

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                              *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
      ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                    (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                        SLOPE STABILITY ANALYSIS SYSTEM
          Modified Bishop, Simplified Janbu, or GLE Method of Slices.
          (Includes Spencer & Morgenstern-Price Type Analysis)
          Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
          Nonlinear Undrained Shear Strength, Curved Phi Envelope,
          Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
          Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/9/2008
Time of Run:                03:56PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7s1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7s1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t7s1a.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 7-7'; CS-3;
                      planar; static
BOUNDARY COORDINATES
   12 Top   Boundaries
   14 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1          0.00      970.00     195.00      970.00         2
    2        195.00      970.00     249.00      998.00         2
    3        249.00      998.00     257.00      998.00         2
    4        257.00      998.00     306.00     1022.00         2
    5        306.00     1022.00     314.00     1022.00         2
    6        314.00     1022.00     360.00     1045.00         2
    7        360.00     1045.00     385.00     1045.00         2
    8        385.00     1045.00     565.00     1094.00         2
    9        565.00     1094.00     600.00     1077.00         2
   10        600.00     1077.00     632.00     1060.00         2
   11        632.00     1060.00     680.00     1070.00         1
   12        680.00     1070.00     700.00     1070.00         1
   13        632.00     1060.00     660.00     1050.00         2
   14        660.00     1050.00     700.00     1036.00         2
User Specified Y-Origin =       800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
  1   125.0    130.0     300.0    30.0    0.00       0.0      0
  2   130.0    135.0     350.0    30.0    0.00       0.0      0
ANISOTROPIC STRENGTH PARAMETERS
    1 soil type(s)
Soil Type  2 Is Anisotropic
Number Of Direction Ranges Specified =  3
Direction    Counterclockwise     Cohesion      Friction
  Range       Direction Limit    Intercept       Angle
   No.            (deg)            (psf)         (deg)
    1              16.5            350.00         30.00
    2              19.5            200.00         20.00
    3              90.0            350.00         30.00
ANISOTROPIC SOIL NOTES:
   (1) An input value of 0.01 for C and/or Phi will cause Aniso
       C and/or Phi to be ignored in that range.
   (2) An input value of 0.02 for Phi will set both Phi and
       C equal to zero, with no water weight in the tension crack.
   (3) An input value of 0.03 for Phi will set both Phi and
```

C equal to zero, with water weight in the tension crack.
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Sliding Block Surfaces, Has Been
Specified.
2000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	150.00	917.00	270.00	968.00	60.00
2	330.00	995.00	565.00	1058.00	60.00

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 2000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 82
Number of Trial Surfaces with Misleading FS = 1
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 4.2 %
Statistical Data On All Valid FS Values:
FS Max = 6.905 FS Min = 1.476 FS Ave = 2.941
Standard Deviation = 0.892 Coefficient of Variation = 30.32 %
((Simplified Janbu FS for Critical Surface = 1.411))
Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	182.18	970.00
2	217.17	959.69
3	499.06	1058.07
4	522.37	1082.40

*** FOS = 1.476 Theta (ki=1.0) = 16.89 ***
Lambda = 0.304
Individual data on the 10 slices

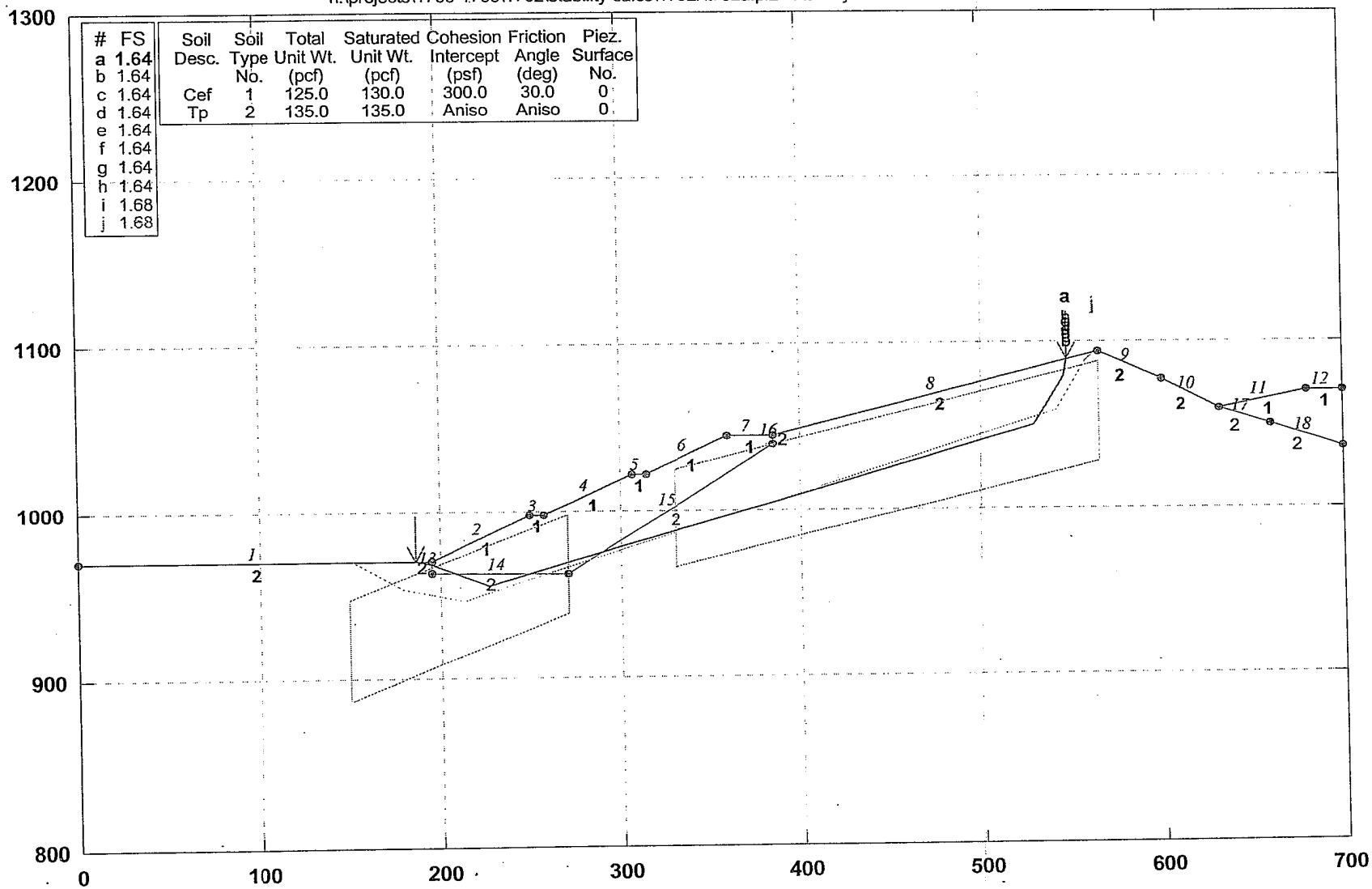
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	12.8	3146.5	0.0	0.0	0.	0.	0.0	0.0	0.0
2	22.2	36859.2	0.0	0.0	0.	0.	0.0	0.0	0.0
3	31.8	101375.2	0.0	0.0	0.	0.	0.0	0.0	0.0
4	8.0	26833.6	0.0	0.0	0.	0.	0.0	0.0	0.0
5	49.0	177437.0	0.0	0.0	0.	0.	0.0	0.0	0.0
6	8.0	31105.1	0.0	0.0	0.	0.	0.0	0.0	0.0
7	46.0	191275.7	0.0	0.0	0.	0.	0.0	0.0	0.0
8	25.0	101063.8	0.0	0.0	0.	0.	0.0	0.0	0.0
9	114.1	331495.9	0.0	0.0	0.	0.	0.0	0.0	0.0
10	23.3	27237.8	0.0	0.0	0.	0.	0.0	0.0	0.0

Lndmrk off-site grading; 7-7'; CS-3; planar; static; buttress w/ 7'dx75'w key

n:\projects\1700-1799\1702\stability calcs\1702r\t7s2a.pl2 Run By: KPC 7/21/2008 02:08PM

#	FS
a	1.64
b	1.64
c	1.64
d	1.64
e	1.64
f	1.64
g	1.64
h	1.64
i	1.68
j	1.68

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0
Tp	2	135.0	135.0	Aniso	Aniso	0



GSTABL7 v.2 FSmin=1.64

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 24

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                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/21/2008
Time of Run:                02:09PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7s2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7s2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t7s2a.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 7-7'; CS-3;
                      planar; static; buttress w/ 7'dx75'w key
BOUNDARY COORDINATES
   12 Top   Boundaries
   18 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)     Below Bnd
    1         0.00      970.00      195.00      970.00        2
    2       195.00      970.00      249.00      998.00        1
    3       249.00      998.00      257.00      998.00        1
    4       257.00      998.00      306.00     1022.00        1
    5       306.00     1022.00      314.00     1022.00        1
    6       314.00     1022.00      360.00     1045.00        1
    7       360.00     1045.00      385.00     1045.00        1
    8       385.00     1045.00      565.00     1094.00        2
    9       565.00     1094.00      600.00     1077.00        2
   10       600.00     1077.00      632.00     1060.00        2
   11       632.00     1060.00      680.00     1070.00        1
   12       680.00     1070.00      700.00     1070.00        1
   13       195.00      970.00      195.01      963.00        2
   14       195.01      963.00      270.00      962.00        2
   15       270.00      962.00      384.99     1040.00        2
   16       384.99     1040.00      385.00     1045.00        2
   17       632.00     1060.00      660.00     1050.00        2
   18       660.00     1050.00      700.00     1036.00        2
 User Specified Y-Origin =        800.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1   125.0    130.0     300.0     30.0    0.00       0.0      0
   2   135.0    135.0     350.0     30.0    0.00       0.0      0
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  2 Is Anisotropic
 Number Of Direction Ranges Specified =  3
 Direction    Counterclockwise     Cohesion      Friction
   Range       Direction Limit    Intercept       Angle
    No.            (deg)            (psf)         (deg)
     1             16.5             350.00         30.00
     2             19.5             200.00         20.00
     3             90.0             350.00         30.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
```

C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 34.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	150.00	917.00	270.00	968.00	60.00
2	330.00	995.00	565.00	1058.00	60.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 24
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 0.8 %
Statistical Data On All Valid FS Values:
FS Max = 12.401 FS Min = 1.638 FS Ave = 3.023
Standard Deviation = 1.078 Coefficient of Variation = 35.67 %
((Simplified Janbu FS for Critical Surface = 1.510))
Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	186.16	970.00
2	194.79	967.63
3	226.28	954.82
4	529.30	1049.82
5	546.40	1079.21
6	547.55	1089.25

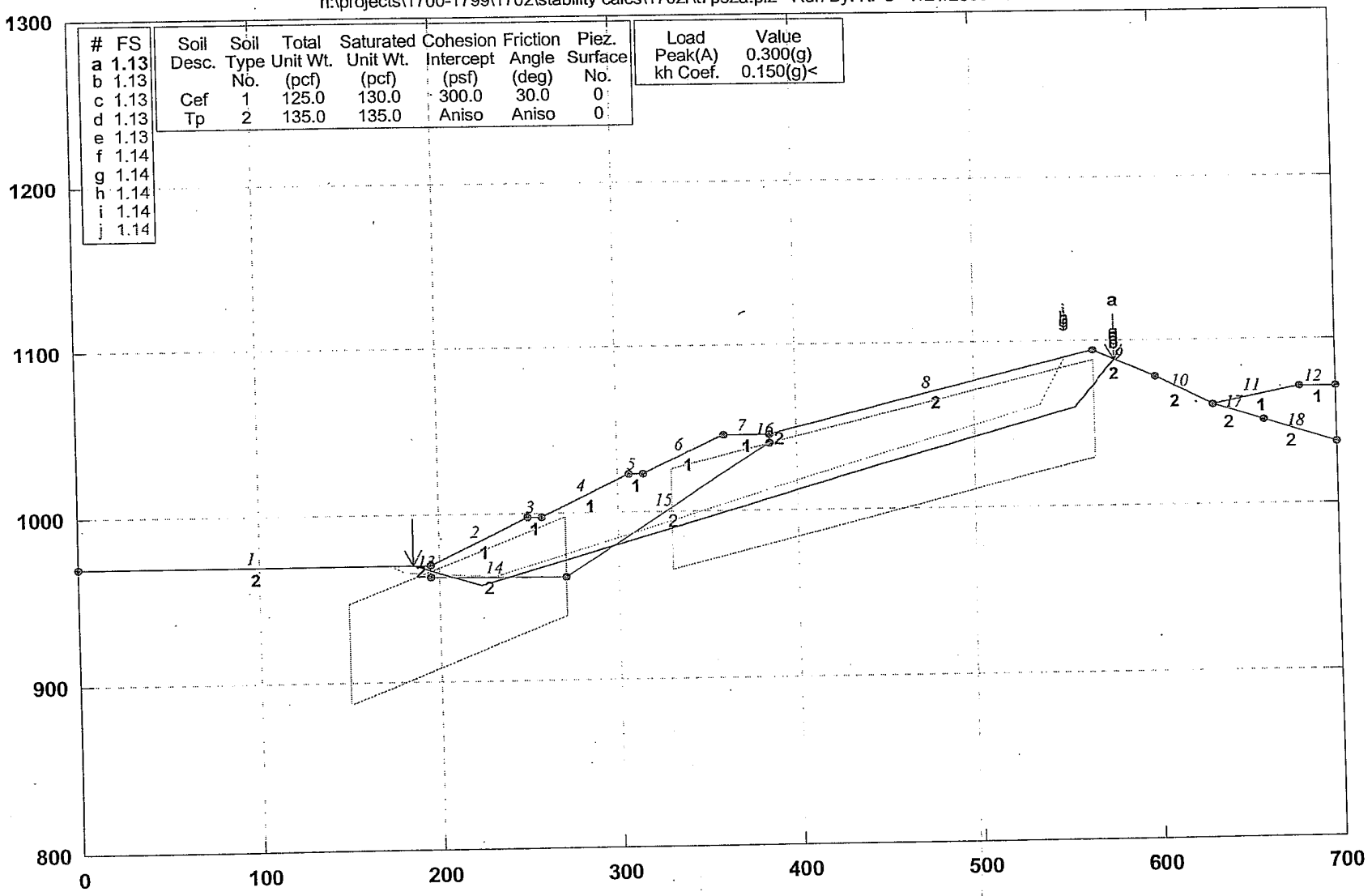
*** FOS = 1.638 Theta (ki=1.0) = 17.13 ***
Lambda = 0.308
Individual data on the 17 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	8.6	1378.9	0.0	0.0	0.	0.	0.0	0.0	0.0
2	0.2	69.2	0.0	0.0	0.	0.	0.0	0.0	0.0
3	0.0	1.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	11.5	11262.5	0.0	0.0	0.	0.	0.0	0.0	0.0
5	19.7	55689.1	0.0	0.0	0.	0.	0.0	0.0	0.0
6	22.7	96699.1	0.0	0.0	0.	0.	0.0	0.0	0.0
7	1.0	4621.8	0.0	0.0	0.	0.	0.0	0.0	0.0
8	7.0	30182.5	0.0	0.0	0.	0.	0.0	0.0	0.0
9	30.9	140057.3	0.0	0.0	0.	0.	0.0	0.0	0.0
10	18.1	92481.0	0.0	0.0	0.	0.	0.0	0.0	0.0
11	8.0	41578.3	0.0	0.0	0.	0.	0.0	0.0	0.0
12	46.0	261060.6	0.0	0.0	0.	0.	0.0	0.0	0.0
13	25.0	146223.2	0.0	0.0	0.	0.	0.0	0.0	0.0

14	0.0	54.4	0.0	0.0	0.	0.	0.0	0.0	0.0
15	144.3	729396.8	0.0	0.0	0.	0.	0.0	0.0	0.0
16	17.1	51005.9	0.0	0.0	0.	0.	0.0	0.0	0.0
17	1.2	756.2	0.0	0.0	0.	0.	0.0	0.0	0.0

Lndmrk off-site grading; 7-7'; CS-3; planar; pseudo; buttress w/ 7'dx75'w key

n:\projects\1700-1799\1702\stability calcs\1702r\t7ps2a.pl2 Run By: KPC 7/21/2008 02:10PM



#	FS
a	1.13
b	1.13
c	1.13
d	1.13
e	1.13
f	1.14
g	1.14
h	1.14
i	1.14
j	1.14

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0
Tp	2	135.0	135.0	Aniso	Aniso	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.13

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 25

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
             (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/21/2008
Time of Run:                02:10PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7ps2a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t7ps2a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t7ps2a.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 7-7'; CS-3;
                      planar; pseudo; buttress w/ 7'dx75'w key

BOUNDARY COORDINATES
   12 Top   Boundaries
   18 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	970.00	195.00	970.00	2
2	195.00	970.00	249.00	998.00	1
3	249.00	998.00	257.00	998.00	1
4	257.00	998.00	306.00	1022.00	1
5	306.00	1022.00	314.00	1022.00	1
6	314.00	1022.00	360.00	1045.00	1
7	360.00	1045.00	385.00	1045.00	2
8	385.00	1045.00	565.00	1094.00	2
9	565.00	1094.00	600.00	1077.00	2
10	600.00	1077.00	632.00	1060.00	2
11	632.00	1060.00	680.00	1070.00	1
12	680.00	1070.00	700.00	1070.00	1
13	195.00	970.00	195.01	963.00	2
14	195.01	963.00	270.00	962.00	2
15	270.00	962.00	384.99	1040.00	2
16	384.99	1040.00	385.00	1045.00	2
17	632.00	1060.00	660.00	1050.00	2
18	660.00	1050.00	700.00	1036.00	2

```
User Specified Y-Origin =        800.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  2 Type(s) of Soil
```

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	300.0	30.0	0.00	0.0	0
2	135.0	135.0	500.0	36.0	0.00	0.0	0

```
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  2 Is Anisotropic
 Number Of Direction Ranges Specified =  3
```

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	16.5	500.00	36.00
2	19.5	200.00	20.00
3	90.0	500.00	36.00

```
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
```

C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
3000 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	150.00	917.00	270.00	968.00	60.00
2	330.00	995.00	565.00	1058.00	60.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 3000
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 129
Number of Trial Surfaces with Misleading FS = 5
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 4.5 %
Statistical Data On All Valid FS Values:
FS Max = 5.951 FS Min = 1.125 FS Ave = 2.315
Standard Deviation = 0.645 Coefficient of Variation = 27.85 %
(Simplified Janbu FS for Critical Surface = 1.057))
Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	185.54	970.00
2	223.12	957.29
3	554.31	1058.93
4	577.19	1088.03

*** FOS = 1.125 Theta (ki=1.0) = 15.79 ***
Lambda = 0.263
Individual data on the 16 slices

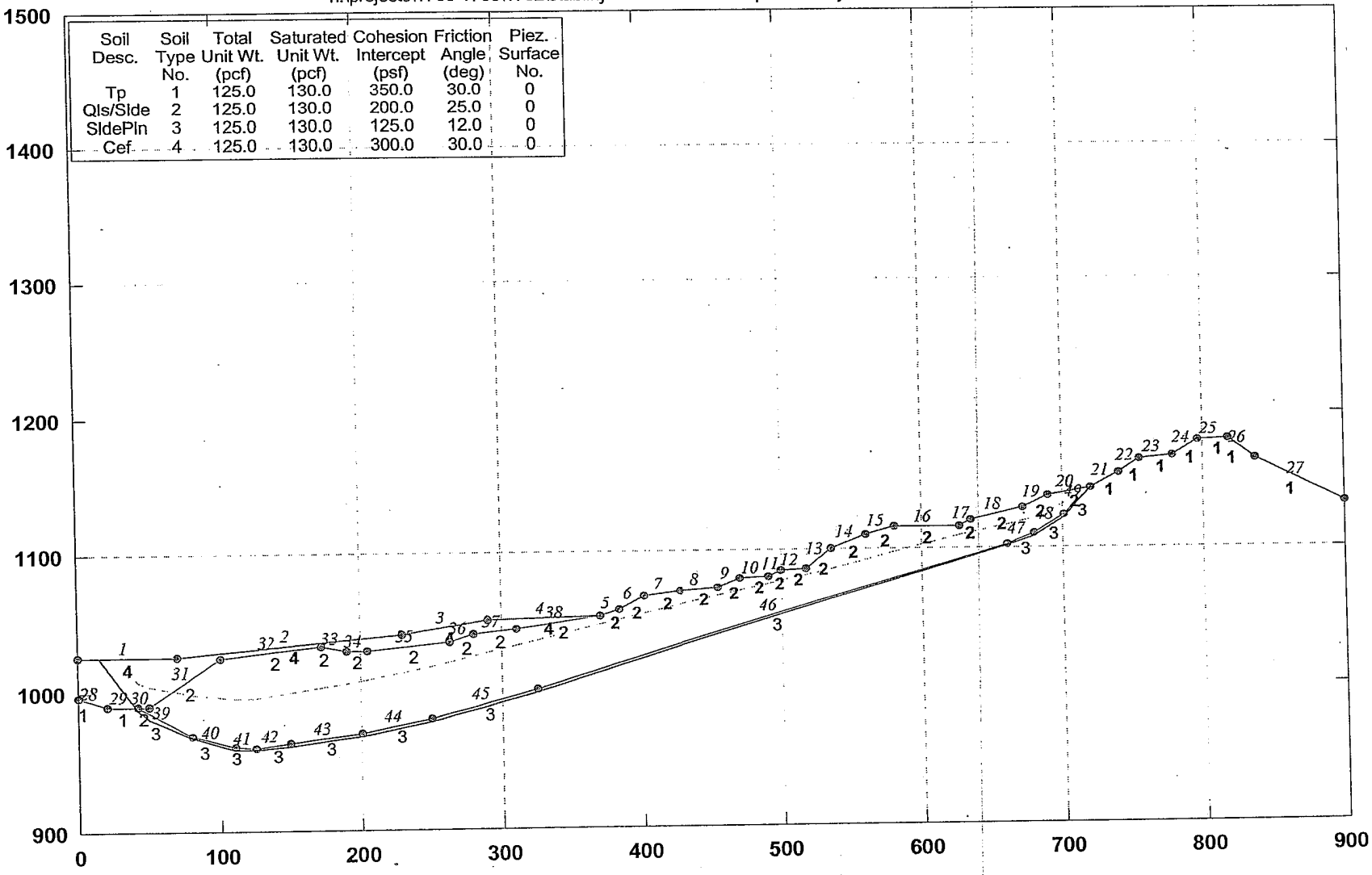
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	9.5	2044.3	0.0	0.0	0.	0.	306.7	0.0	0.0
2	0.0	1.9	0.0	0.0	0.	0.	0.3	0.0	0.0
3	11.7	11990.7	0.0	0.0	0.	0.	1798.6	0.0	0.0
4	16.4	42037.5	0.0	0.0	0.	0.	6305.6	0.0	0.0
5	16.7	61017.6	0.0	0.0	0.	0.	9152.6	0.0	0.0
6	9.2	36593.4	0.0	0.0	0.	0.	5489.0	0.0	0.0
7	8.0	31544.2	0.0	0.0	0.	0.	4731.6	0.0	0.0
8	39.0	165378.6	0.0	0.0	0.	0.	24806.8	0.0	0.0
9	10.0	47943.1	0.0	0.0	0.	0.	7191.5	0.0	0.0
10	8.0	38466.4	0.0	0.0	0.	0.	5770.0	0.0	0.0

11	46.0	244275.4	0.0	0.0	0.	0.	36641.3	0.0	0.0
12	25.0	137896.0	0.0	0.0	0.	0.	20684.4	0.0	0.0
13	0.0	51.1	0.0	0.0	0.	0.	7.7	0.0	0.0
14	169.3	802300.7	0.0	0.0	0.	0.	*******	0.0	0.0
15	10.7	38679.2	0.0	0.0	0.	0.	5801.9	0.0	0.0
16	12.2	17663.8	0.0	0.0	0.	0.	2649.6	0.0	0.0

Lndmrk off-site grading; 8-8'; Qls-XIV(B); static; basal Qls plane - no removals

n:\projects\1700-1799\1702\stability calcs\1702r\t8s1a.plt Run By: MJG 7/17/2008 03:57PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tp	1	125.0	130.0	350.0	30.0	0
Qls/Slde	2	125.0	130.0	200.0	25.0	0
SldePln	3	125.0	130.0	125.0	12.0	0
Cef	4	125.0	130.0	300.0	30.0	0



GSTABL7 v.2 FSmin=2.40

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 26

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:        7/17/2008
Time of Run:              03:57PM
Run By:                   MJG
Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\t8s1a.in
Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\t8s1a.OUT
Unit System:              English
Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t8s1a.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 8-8'; Qls-XIV(B
                      ); static; basal Qls plane - no removals

BOUNDARY COORDINATES
   27 Top   Boundaries
   49 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	1025.00	70.00	1025.00	4
2	70.00	1025.00	230.00	1040.00	4
3	230.00	1040.00	290.00	1050.00	4
4	290.00	1050.00	370.00	1052.00	4
5	370.00	1052.00	384.00	1056.00	2
6	384.00	1056.00	402.00	1066.00	2
7	402.00	1066.00	428.00	1070.00	2
8	428.00	1070.00	455.00	1072.00	2
9	455.00	1072.00	470.00	1079.00	2
10	470.00	1079.00	490.00	1080.00	2
11	490.00	1080.00	500.00	1085.00	2
12	500.00	1085.00	517.00	1086.00	2
13	517.00	1086.00	535.00	1100.00	2
14	535.00	1100.00	560.00	1110.00	2
15	560.00	1110.00	580.00	1116.00	2
16	580.00	1116.00	627.00	1116.00	2
17	627.00	1116.00	634.00	1120.00	2
18	634.00	1120.00	672.00	1130.00	2
19	672.00	1130.00	690.00	1138.00	2
20	690.00	1138.00	720.00	1144.00	2
21	720.00	1144.00	740.00	1155.00	1
22	740.00	1155.00	755.00	1166.00	1
23	755.00	1166.00	779.00	1168.00	1
24	779.00	1168.00	796.00	1179.00	1
25	796.00	1179.00	818.00	1180.00	1
26	818.00	1180.00	837.00	1165.00	1
27	837.00	1165.00	900.00	1133.00	1
28	0.00	997.00	20.00	990.00	1
29	20.00	990.00	42.00	990.00	1
30	42.00	990.00	50.00	990.00	2
31	50.00	990.00	100.00	1024.00	2
32	100.00	1024.00	172.00	1032.00	2
33	172.00	1032.00	190.00	1028.00	2
34	190.00	1028.00	205.00	1028.00	2
35	205.00	1028.00	263.00	1034.00	2
36	263.00	1034.00	280.00	1039.00	2
37	280.00	1039.00	310.00	1043.00	2
38	310.00	1043.00	370.00	1052.00	2
39	42.00	990.00	80.00	969.00	3
40	80.00	969.00	110.00	961.00	3

41	110.00	961.00	125.00	960.00	3
42	125.00	960.00	150.00	963.00	3
43	150.00	963.00	200.00	970.00	3
44	200.00	970.00	250.00	980.00	3
45	250.00	980.00	325.00	1000.00	3
46	325.00	1000.00	660.00	1103.00	3
47	660.00	1103.00	680.00	1110.00	3
48	680.00	1110.00	701.00	1124.00	3
49	701.00	1124.00	720.00	1144.00	3

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	350.0	30.0	0.00	0.0	0
2	125.0	130.0	200.0	25.0	0.00	0.0	0
3	125.0	130.0	125.0	12.0	0.00	0.0	0
4	125.0	130.0	300.0	30.0	0.00	0.0	0

Trial Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	15.00	1025.00
2	42.00	988.00
3	50.00	981.00
4	80.00	967.00
5	110.00	959.00
6	125.00	958.00
7	150.00	961.00
8	200.00	968.00
9	250.00	978.00
10	325.00	998.00
11	660.00	1101.00
12	680.00	1108.00
13	701.00	1122.00
14	720.00	1144.00

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
9.00	2.146	2.544	0.158
13.50	0.001	3.040	0.240
8.10	2.296	2.469	0.142
5.83	2.556	2.308	0.102
6.82	2.451	2.374	0.120
7.41	2.379	2.416	0.130
7.15	2.411	2.398	0.125
7.22	2.403	2.403	0.127

((Simplified Janbu FS for Specified Surface = 2.008))
Factor Of Safety For The Preceding Specified Surface = 2.403
Theta (ki = 1.0) = 7.22 Deg Lambda = 0.127
Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails
(if applicable) have been applied to the slice base(s)
on which they intersect.
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
Tension Crack Water Force = 0.00(lbs)
Specified Tension Crack Water Force Factor = 0.000
Depth of Tension Crack (zo) at Center of Last Slice = 0.905(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/R	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	40.54	1009.12	0.546	214205.	1.000	7.22	27134.9
2	42.00	1007.25	0.520	238376.	1.000	7.22	30196.9
3	50.00	1004.23	0.528	288434.	1.000	7.22	36538.1

4	70.00	1000.55	0.542	366506.	1.000	7.22	46428.1
5	80.00	998.34	0.532	411423.	1.000	7.22	52118.1
6	100.00	996.30	0.524	472722.	1.000	7.22	59883.2
7	110.00	995.16	0.518	505968.	1.000	7.22	64094.8
8	125.00	995.48	0.519	528128.	1.000	7.22	66902.0
9	150.00	999.06	0.532	522408.	1.000	7.22	66177.4
10	172.00	1002.50	0.545	513611.	1.000	7.22	65063.0
11	190.00	1005.31	0.556	506519.	1.000	7.22	64164.6
12	200.00	1006.88	0.562	502621.	1.000	7.22	63670.7
13	205.00	1007.86	0.566	498192.	1.000	7.22	63109.7
14	230.00	1012.75	0.587	476590.	1.000	7.22	60373.2
15	250.00	1016.67	0.592	459760.	1.000	7.22	58241.3
16	263.00	1019.80	0.599	442412.	1.000	7.22	56043.6
17	280.00	1023.91	0.608	420272.	1.000	7.22	53239.0
18	290.00	1026.34	0.614	407539.	1.000	7.22	51626.0
19	310.00	1031.16	0.658	383323.	1.000	7.22	48558.5
20	325.00	1034.72	0.694	366524.	1.000	7.22	46430.3
21	370.00	1045.85	0.847	314218.	1.000	7.22	39804.4
22	384.00	1049.24	0.828	300329.	1.000	7.22	38044.9
23	402.00	1053.52	0.718	281491.	1.000	7.22	35658.6
24	428.00	1059.99	0.752	254120.	1.000	7.22	32191.3
25	455.00	1066.43	0.836	229333.	1.000	7.22	29051.3
26	470.00	1069.95	0.752	216329.	1.000	7.22	27404.1
27	490.00	1074.62	0.828	199714.	1.000	7.22	25299.2
28	500.00	1076.91	0.756	191828.	1.000	7.22	24300.2
29	517.00	1080.76	0.819	178933.	1.000	7.22	22666.8
30	535.00	1085.01	0.600	164282.	1.000	7.22	20810.8
31	560.00	1091.61	0.537	140408.	1.000	7.22	17786.6
32	580.00	1097.32	0.528	120753.	0.993	7.17	15080.5
33	627.00	1109.45	0.739	83884.	0.660	4.78	4622.8
34	634.00	1110.97	0.666	79537.	0.610	4.42	3748.2
35	660.00	1116.55	0.602	63111.	0.426	3.09	1447.7
36	672.00	1119.33	0.570	54382.	0.340	2.47	798.4
37	680.00	1121.18	0.516	48581.	0.284	2.06	495.3
38	690.00	1126.20	0.494	33055.	0.213	1.54	189.6
39	701.00	1132.71	0.589	18549.	0.135	0.98	42.7
40	720.00	1314.67	0.000	-13.	0.000	0.00	0.0

Table 1 - Individual data on the 40 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	25.5	55369.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	1.5	6567.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	8.0	40500.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	20.0	121666.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	10.0	70169.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	20.0	156354.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	10.0	84934.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	15.0	133037.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	25.0	224452.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	22.0	195226.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	18.0	157649.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	10.0	86773.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	5.0	43076.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	25.0	210400.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	20.0	164166.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	13.0	105110.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	17.0	134264.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	10.0	77291.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	20.0	147291.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	15.0	102539.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	45.0	261672.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	14.0	70020.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	18.0	94708.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	26.0	137567.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	27.0	125484.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0

26	15.0	66044.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	20.0	84608.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	10.0	40289.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	17.0	66046.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	18.0	74699.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	25.0	120592.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	20.0	99179.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
33	47.0	190183.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
34	7.0	22821.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	26.0	85858.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
36	12.0	37981.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	8.0	25177.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
38	10.0	30555.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	11.0	28554.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
40	19.0	21612.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the 40 Slices -

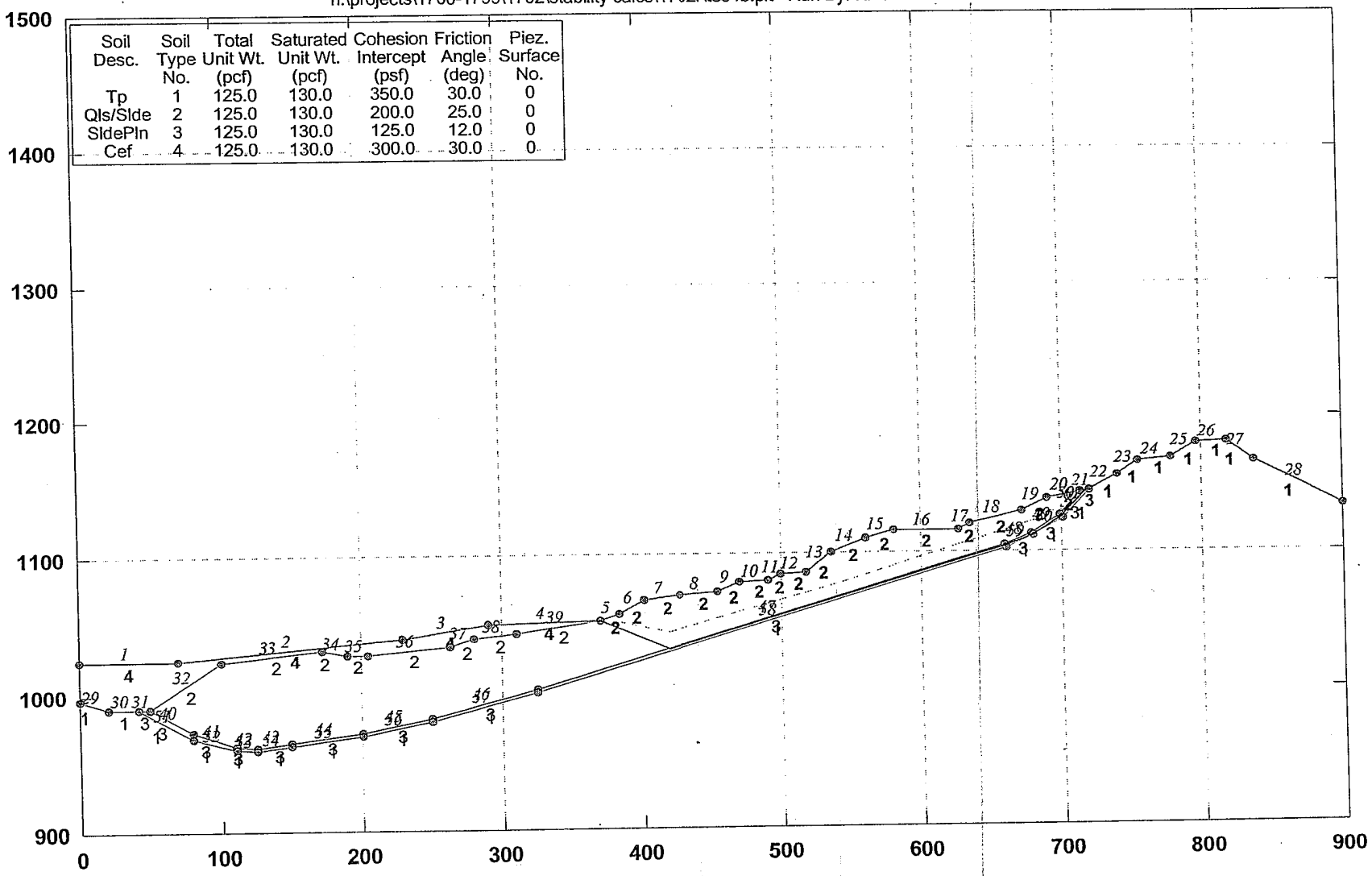
Slice No.	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	-53.88	27.77	43.33	5088.22	1289.46	1347.61
2	-53.88	41.27	2.48	10110.49	2652.61	2575.31
3	-41.19	46.00	10.63	6388.98	3809.92	617.27
4	-25.02	60.00	22.07	6882.35	5512.62	660.92
5	-25.02	75.00	11.04	7933.23	6358.62	753.89
6	-14.93	90.00	20.70	8415.30	7553.75	796.54
7	-14.93	105.00	10.35	9140.84	8206.71	860.73
8	-3.81	117.50	15.03	9112.02	8849.50	858.18
9	6.84	137.50	25.18	8849.57	8814.56	834.96
10	7.97	161.00	22.21	8708.51	8788.20	822.48
11	7.97	181.00	18.18	8595.02	8673.69	812.44
12	7.97	195.00	10.10	8515.59	8593.54	805.41
13	11.31	202.50	5.10	8345.83	8447.93	790.40
14	11.31	217.50	25.50	8152.76	8252.58	773.31
15	11.31	240.00	20.40	7951.48	8048.93	755.51
16	14.93	256.50	13.45	7721.65	7812.42	735.17
17	14.93	271.50	17.59	7542.43	7631.24	719.32
18	14.93	285.00	10.35	7381.13	7468.19	705.05
19	14.93	300.00	20.70	7032.64	7115.92	674.22
20	14.93	317.50	15.52	6527.32	6605.12	629.51
21	17.09	347.50	47.08	5503.18	5558.16	538.90
22	17.09	377.00	14.65	4732.09	4780.63	470.68
23	17.09	393.00	18.83	4978.62	5029.22	492.49
24	17.09	415.00	27.20	5006.56	5057.40	494.97
25	17.09	441.50	28.25	4396.60	4442.33	441.00
26	17.09	462.50	15.69	4164.75	4208.55	420.49
27	17.09	480.00	20.92	4001.17	4043.60	406.02
28	17.09	495.00	10.46	3810.17	3851.00	389.12
29	17.09	508.50	17.79	3673.81	3713.51	377.06
30	17.09	526.00	18.83	3924.94	3966.73	399.27
31	17.09	547.50	26.15	4563.56	4610.69	455.77
32	17.09	570.00	20.92	4686.67	4739.97	466.66
33	17.09	603.50	49.17	3756.25	3867.76	384.35
34	17.09	630.50	7.32	3037.67	3114.87	320.77
35	17.09	647.00	27.20	3097.02	3156.44	326.03
36	19.29	666.00	12.71	2967.20	2987.44	314.54
37	19.29	676.00	8.48	2962.50	2970.54	314.12
38	33.69	685.00	12.02	2771.96	2542.38	297.27
39	33.69	695.50	13.22	2369.22	2159.86	261.64
40	49.18	710.50	29.07	962.04	743.49	137.14

**** END OF GSTABL7 OUTPUT ****

Lndmrk off-site grading; 8-8'; Qls-XIV(B); static; basal Qls plane - no removals

n:\projects\1700-1799\1702\stability calcs\1702r\t8s4b.plt Run By: KPC 7/21/2008 09:04AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tp	1	125.0	130.0	350.0	30.0	0
Qls/Slde	2	125.0	130.0	200.0	25.0	0
SldePln	3	125.0	130.0	125.0	12.0	0
Cef	4	125.0	130.0	300.0	30.0	0



GSTABL7 v.2 FSmin=1.43

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 27

*** GSTABL7 ***
** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
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SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

Analysis Run Date: 7/21/2008
Time of Run: 09:04AM
Run By: KPC
Input Data Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t8s4b.in
Output Filename: N:\Projects\1700-1799\1702\Stability Calcs\1702R\t8s4b.OUT
Unit System: English
Plotted Output Filename: N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t8s4b.PLT
PROBLEM DESCRIPTION: Landmark off-site grading; 8-8'; Qls-XIV(B
); static; basal Qls plane - no removals

BOUNDARY COORDINATES
28 Top Boundaries
61 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	1025.00	70.00	1025.00	4
2	70.00	1025.00	230.00	1040.00	4
3	230.00	1040.00	290.00	1050.00	4
4	290.00	1050.00	370.00	1052.00	4
5	370.00	1052.00	384.00	1056.00	2
6	384.00	1056.00	402.00	1066.00	2
7	402.00	1066.00	428.00	1070.00	2
8	428.00	1070.00	455.00	1072.00	2
9	455.00	1072.00	470.00	1079.00	2
10	470.00	1079.00	490.00	1080.00	2
11	490.00	1080.00	500.00	1085.00	2
12	500.00	1085.00	517.00	1086.00	2
13	517.00	1086.00	535.00	1100.00	2
14	535.00	1100.00	560.00	1110.00	2
15	560.00	1110.00	580.00	1116.00	2
16	580.00	1116.00	627.00	1116.00	2
17	627.00	1116.00	634.00	1120.00	2
18	634.00	1120.00	672.00	1130.00	2
19	672.00	1130.00	690.00	1138.00	2
20	690.00	1138.00	713.00	1143.00	2
21	713.00	1143.00	720.00	1144.00	3
22	720.00	1144.00	740.00	1155.00	1
23	740.00	1155.00	755.00	1166.00	1
24	755.00	1166.00	779.00	1168.00	1
25	779.00	1168.00	796.00	1179.00	1
26	796.00	1179.00	818.00	1180.00	1
27	818.00	1180.00	837.00	1165.00	1
28	837.00	1165.00	900.00	1133.00	1
29	0.00	997.00	20.00	990.00	1
30	20.00	990.00	42.00	990.00	1
31	42.00	990.00	50.00	990.00	3
32	50.00	990.00	100.00	1024.00	2
33	100.00	1024.00	172.00	1032.00	2
34	172.00	1032.00	190.00	1028.00	2
35	190.00	1028.00	205.00	1028.00	2
36	205.00	1028.00	263.00	1034.00	2
37	263.00	1034.00	280.00	1039.00	2
38	280.00	1039.00	310.00	1043.00	2
39	310.00	1043.00	370.00	1052.00	2
40	50.00	990.00	80.00	973.00	3

41	80.00	973.00	110.00	963.00	3
42	110.00	963.00	125.00	962.00	3
43	125.00	962.00	150.00	965.00	3
44	150.00	965.00	200.00	972.00	3
45	200.00	972.00	250.00	982.00	3
46	250.00	982.00	325.00	1002.00	3
47	325.00	1002.00	659.00	1105.00	3
48	659.00	1105.00	678.00	1113.00	3
49	678.00	1113.00	699.00	1126.00	3
50	699.00	1126.00	713.00	1143.00	3
51	42.00	990.00	80.00	969.00	1
52	80.00	969.00	110.00	961.00	1
53	110.00	961.00	125.00	960.00	1
54	125.00	960.00	150.00	963.00	1
55	150.00	963.00	200.00	970.00	1
56	200.00	970.00	250.00	980.00	1
57	250.00	980.00	325.00	1000.00	1
58	325.00	1000.00	660.00	1103.00	1
59	660.00	1103.00	680.00	1111.00	1
60	680.00	1111.00	701.00	1124.00	1
61	701.00	1124.00	720.00	1144.00	1

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	350.0	30.0	0.00	0.0	0
2	125.0	130.0	200.0	25.0	0.00	0.0	0
3	125.0	130.0	125.0	12.0	0.00	0.0	0
4	125.0	130.0	300.0	30.0	0.00	0.0	0

Trial Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	370.00	1052.00
2	420.00	1031.00
3	659.50	1104.00
4	679.00	1112.00
5	700.00	1125.00
6	716.50	1143.50

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
9.00	1.560	1.354	0.158
13.50	1.433	1.424	0.240
13.76	1.422	1.429	0.245
13.64	1.427	1.427	0.243

((Simplified Janbu FS for Specified Surface = 1.245))
Factor Of Safety For The Preceding Specified Surface = 1.427
Theta (ki = 1.0) = 13.64 Deg Lambda = 0.243
Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails (if applicable) have been applied to the slice base(s) on which they intersect.
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
Tension Crack Water Force = 0.00(lbs)
Specified Tension Crack Water Force Factor = 0.000
Depth of Tension Crack (zo) at Center of Last Slice = 1.631(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	384.00	1050.76	0.470	13236.	1.000	13.64	3212.6
2	402.00	1046.68	0.296	65067.	1.000	13.64	15793.2
3	419.59	1043.05	0.317	150022.	1.000	13.64	36413.4

4	420.00	1043.03	0.319	151478.	1.000	13.64	36767.0
5	428.00	1045.36	0.326	146705.	1.000	13.64	35608.3
6	455.00	1053.11	0.377	132451.	1.000	13.64	32148.7
7	470.00	1057.40	0.341	125060.	1.000	13.64	30354.8
8	490.00	1063.10	0.389	115699.	1.000	13.64	28082.7
9	500.00	1065.94	0.356	111310.	1.000	13.64	27017.2
10	517.00	1070.75	0.401	104198.	1.000	13.64	25291.1
11	535.00	1075.95	0.292	95947.	1.000	13.64	23288.3
12	560.00	1083.57	0.273	81967.	1.000	13.64	19895.1
13	580.00	1089.96	0.281	70361.	1.000	13.64	17078.0
14	627.00	1105.08	0.502	49356.	1.000	13.64	11979.8
15	634.00	1107.31	0.466	47035.	1.000	13.64	11416.3
16	659.00	1115.06	0.493	38446.	0.830	11.39	6424.4
17	659.50	1115.20	0.493	38279.	0.823	11.29	6285.6
18	672.00	1119.50	0.497	31390.	0.642	8.86	3141.6
19	678.00	1121.42	0.466	28235.	0.556	7.68	2115.2
20	679.00	1121.73	0.461	27706.	0.541	7.48	1969.1
21	690.00	1126.91	0.422	17691.	0.382	5.30	627.9
22	699.00	1131.38	0.449	10640.	0.253	3.51	164.7
23	700.00	1131.90	0.454	9954.	0.238	3.31	134.0
24	713.00	1127.07	-3.652	30.	0.051	0.70	0.0
25	716.50	1597.46	0.000	-1.	0.000	0.00	0.0

Table 1 - Individual data on the 25 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	14.0	8644.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	18.0	41985.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	17.6	71444.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.4	1915.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	8.0	37165.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	27.0	112882.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	15.0	59148.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	20.0	75529.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	10.0	35799.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	17.0	58490.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	18.0	66804.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	25.0	109805.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	20.0	90699.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	47.0	170779.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	7.0	19984.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	25.0	72662.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.5	1420.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	12.5	34048.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	6.0	15730.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	1.0	2636.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	11.0	27707.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	9.0	19555.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	1.0	1921.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	13.0	15111.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	3.5	749.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the 25 Slices -

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	-22.78	377.00	15.18	1042.47	569.32	480.90
2	-22.78	393.00	19.52	3559.15	2150.53	1303.44
3	-22.78	410.80	19.08	6095.59	3744.15	2132.45
4	-22.78	419.80	0.44	5960.79	4339.36	975.66
5	16.95	424.00	8.36	4283.70	4443.83	725.81
6	16.95	441.50	28.23	3854.59	3999.20	661.98
7	16.95	462.50	15.68	3635.24	3771.91	629.20
8	16.95	480.00	20.91	3481.30	3612.41	606.26
9	16.95	495.00	10.45	3299.89	3424.45	579.24
10	16.95	508.50	17.77	3171.24	3291.14	560.07
11	16.95	526.00	18.82	3421.19	3550.13	597.31

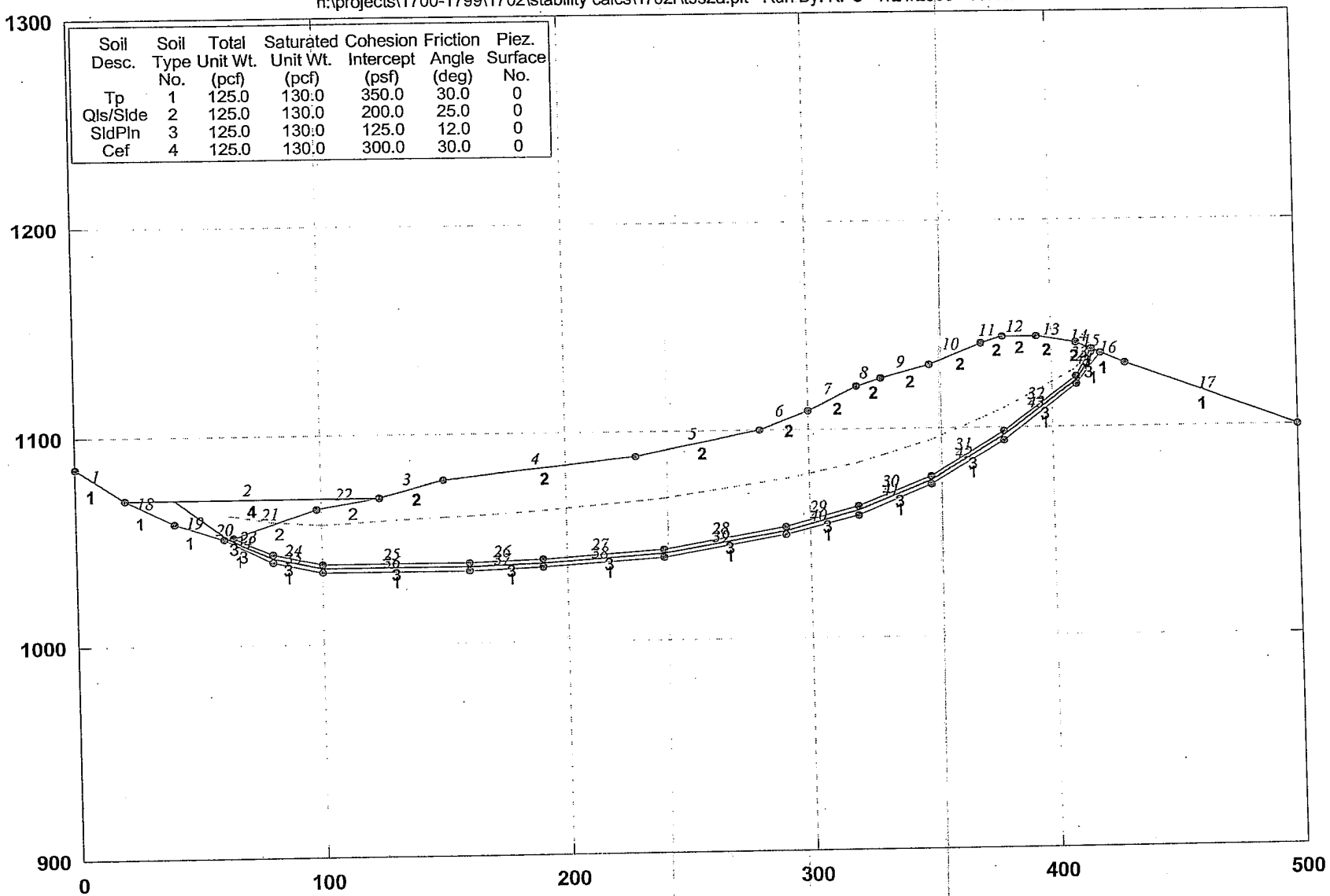
12	16.95	547.50	26.14	4049.73	4201.40	690.95
13	16.95	570.00	20.91	4181.50	4337.94	710.58
14	16.95	603.50	49.13	3349.39	3475.74	586.61
15	16.95	630.50	7.32	2630.52	2730.87	479.51
16	16.95	646.50	26.14	2620.62	2780.20	478.04
17	16.95	659.25	0.52	2507.06	2716.72	461.12
18	22.31	665.75	13.51	2332.57	2520.06	435.12
19	22.31	675.00	6.49	2270.29	2425.60	425.84
20	22.31	678.50	1.08	2295.26	2439.44	429.56
21	31.76	684.50	12.94	2092.28	2141.69	399.32
22	31.76	694.50	10.56	1839.65	1847.53	361.69
23	31.76	699.50	1.18	1639.71	1634.08	331.90
24	48.27	706.50	19.53	874.01	773.70	217.82
25	48.27	714.75	5.26	99.13	142.45	102.38

**** END OF GSTABL7 OUTPUT ****

Lndmrk off-site grading; 9-9'; Qls-XIII(B); static; Qls plane - no removals

n:\projects\1700-1799\1702\stability calcs\1702r\t9s2a.plt Run By: KPC 7/21/2008 08:41AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Tp	1	125.0	130.0	350.0	30.0	0
Qls/Side	2	125.0	130.0	200.0	25.0	0
SldPln	3	125.0	130.0	125.0	12.0	0
Cef	4	125.0	130.0	300.0	30.0	0



GSTABL7 v.2 FSmin=1.42

Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)

GSTABL7

Run No. 28

```
                    *** GSTABL7 ***
              ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
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Analysis Run Date:        7/21/2008
Time of Run:              08:41AM
Run By:                   KPC
Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\t9s2a.in
Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\t9s2a.OUT
Unit System:              English
Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t9s2a.PLT
PROBLEM DESCRIPTION:      Lndmrk off-site grading; 9-9'; Qls-XIII(
                          B); static; Qls plane - no removals

BOUNDARY COORDINATES
   17 Top   Boundaries
   44 Total Boundaries
```

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	1085.00	20.00	1070.00	1
2	20.00	1070.00	124.00	1070.00	4
3	124.00	1070.00	150.00	1078.00	2
4	150.00	1078.00	229.00	1088.00	2
5	229.00	1088.00	280.00	1100.00	2
6	280.00	1100.00	300.00	1109.00	2
7	300.00	1109.00	320.00	1120.00	2
8	320.00	1120.00	330.00	1124.00	2
9	330.00	1124.00	350.00	1130.00	2
10	350.00	1130.00	371.00	1140.00	2
11	371.00	1140.00	380.00	1143.00	2
12	380.00	1143.00	394.00	1143.00	2
13	394.00	1143.00	410.00	1140.00	2
14	410.00	1140.00	416.00	1137.00	2
15	416.00	1137.00	420.00	1135.00	3
16	420.00	1135.00	430.00	1130.00	1
17	430.00	1130.00	500.00	1100.00	1
18	20.00	1070.00	40.00	1059.00	1
19	40.00	1059.00	60.00	1051.00	1
20	60.00	1051.00	64.00	1052.00	3
21	64.00	1052.00	98.00	1065.00	2
22	98.00	1065.00	124.00	1070.00	2
23	64.00	1052.00	80.00	1044.00	3
24	80.00	1044.00	100.00	1039.00	3
25	100.00	1039.00	160.00	1039.00	3
26	160.00	1039.00	190.00	1040.00	3
27	190.00	1040.00	240.00	1044.00	3
28	240.00	1044.00	290.00	1054.00	3
29	290.00	1054.00	320.00	1063.00	3
30	320.00	1063.00	350.00	1077.00	3
31	350.00	1077.00	380.00	1098.00	3
32	380.00	1098.00	410.00	1124.00	3
33	410.00	1124.00	416.00	1137.00	3
34	60.00	1051.00	80.00	1040.00	1
35	80.00	1040.00	100.00	1035.00	1
36	100.00	1035.00	160.00	1035.00	1
37	160.00	1035.00	190.00	1036.00	1
38	190.00	1036.00	240.00	1040.00	1
39	240.00	1040.00	290.00	1050.00	1
40	290.00	1050.00	320.00	1059.00	1

41	320.00	1059.00	350.00	1073.00	1
42	350.00	1073.00	380.00	1094.00	1
43	380.00	1094.00	410.00	1120.00	1
44	410.00	1120.00	420.00	1135.00	1

User Specified Y-Origin = 900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
4 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	125.0	130.0	350.0	30.0	0.00	0.0	0
2	125.0	130.0	200.0	25.0	0.00	0.0	0
3	125.0	130.0	125.0	12.0	0.00	0.0	0
4	125.0	130.0	300.0	30.0	0.00	0.0	0

Trial Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	40.00	1070.00
2	62.00	1051.50
3	80.00	1042.00
4	100.00	1037.00
5	160.00	1037.00
6	190.00	1038.00
7	240.00	1042.00
8	290.00	1052.00
9	320.00	1061.00
10	350.00	1075.00
11	380.00	1096.00
12	410.00	1122.00
13	418.00	1136.00

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
9.00	1.394	1.431	0.158
13.50	1.063	1.546	0.240
8.51	1.421	1.420	0.150
6.87	1.507	1.385	0.120
8.53	1.420	1.420	0.150

((Simplified Janbu FS for Specified Surface = 1.267))
Factor Of Safety For The Preceding Specified Surface = 1.420
Theta (ki = 1.0) = 8.53 Deg Lambda = 0.150
Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails (if applicable) have been applied to the slice base(s) on which they intersect.
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
Tension Crack Water Force = 0.00(lbs)
Specified Tension Crack Water Force Factor = 0.000
Depth of Tension Crack (zo) at Center of Last Slice = 1.527(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	62.00	1062.40	0.589	85159.	1.000	8.53	12773.0
2	64.00	1062.15	0.599	89414.	1.000	8.53	13411.1
3	80.00	1059.08	0.610	131394.	1.000	8.53	19707.7
4	98.00	1057.69	0.621	163755.	1.000	8.53	24561.5
5	100.00	1057.51	0.622	167630.	1.000	8.53	25142.8
6	124.00	1059.00	0.667	185144.	1.000	8.53	27769.6
7	150.00	1060.46	0.572	206129.	1.000	8.53	30917.3
8	160.00	1060.95	0.567	215097.	1.000	8.53	32262.4
9	190.00	1063.05	0.556	237312.	1.000	8.53	35594.3
10	229.00	1066.92	0.550	256567.	1.000	8.53	38482.4
11	240.00	1068.01	0.535	262168.	1.000	8.53	39322.5
12	280.00	1074.85	0.497	253649.	1.000	8.53	38044.8

13	290.00	1076.57	0.468	251399.	1.000	8.53	37707.3
14	300.00	1078.91	0.443	242830.	1.000	8.53	36422.0
15	320.00	1083.73	0.385	224534.	1.000	8.53	33677.9
16	330.00	1087.31	0.371	204468.	1.000	8.53	30668.2
17	350.00	1094.37	0.352	166132.	0.899	7.68	20160.0
18	371.00	1104.50	0.294	104953.	0.622	5.33	6084.3
19	380.00	1108.93	0.275	80377.	0.503	4.31	3045.9
20	394.00	1117.49	0.268	39290.	0.317	2.73	593.9
21	410.00	1127.44	0.302	8973.	0.106	0.91	15.1
22	416.00	1141.19	1.931	152.	0.026	0.23	0.0
23	418.00	1295.07	0.000	7.	0.000	0.00	0.0

Table 1 - Individual data on the 23 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	22.0	25437.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	2.0	4757.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	16.0	47555.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	18.0	68062.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	2.0	8187.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	24.0	99000.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	26.0	120250.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	10.0	52041.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	30.0	163742.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	39.0	224111.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	11.0	65634.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	40.0	246470.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	10.0	64062.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	10.0	66562.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	20.0	141250.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	10.0	73333.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	20.0	141666.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	21.0	138206.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	9.0	54731.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	14.0	71633.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	16.0	52866.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	6.0	8437.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	2.0	562.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the 23 Slices -

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	-40.06	51.00	28.74	2899.28	884.95	1390.00
2	-27.82	63.00	2.26	2975.55	2103.49	533.41
3	-27.82	72.00	18.09	3700.18	2628.59	641.87
4	-14.04	89.00	18.55	4228.14	3668.35	720.89
5	-14.04	99.00	2.06	4574.36	3971.52	772.72
6	0.00	112.00	24.00	4233.24	4125.00	721.66
7	0.00	137.00	26.00	4744.72	4625.00	798.22
8	0.00	155.00	10.00	5337.14	5204.12	886.89
9	1.91	175.00	30.02	5537.34	5455.04	916.86
10	4.57	209.50	39.12	5743.86	5728.14	947.77
11	4.57	234.50	11.04	5963.83	5947.75	980.69
12	11.31	260.00	40.79	5934.90	6042.10	976.36
13	11.31	285.00	10.20	6170.55	6281.84	1011.64
14	16.70	295.00	10.44	6223.28	6375.53	1019.53
15	16.70	310.00	20.88	6603.86	6764.65	1076.49
16	25.02	325.00	11.04	6537.93	6645.34	1066.62
17	25.02	340.00	22.07	6203.11	6418.79	1016.51
18	34.99	360.50	25.63	5363.93	5391.57	890.90
19	34.99	375.50	10.99	5076.44	4981.96	847.87
20	40.91	387.00	18.53	4197.71	3866.61	716.34
21	40.91	402.00	21.17	2761.73	2496.92	501.40
22	60.26	413.00	12.09	973.56	697.70	233.75
23	60.26	417.00	4.03	100.56	139.54	103.08

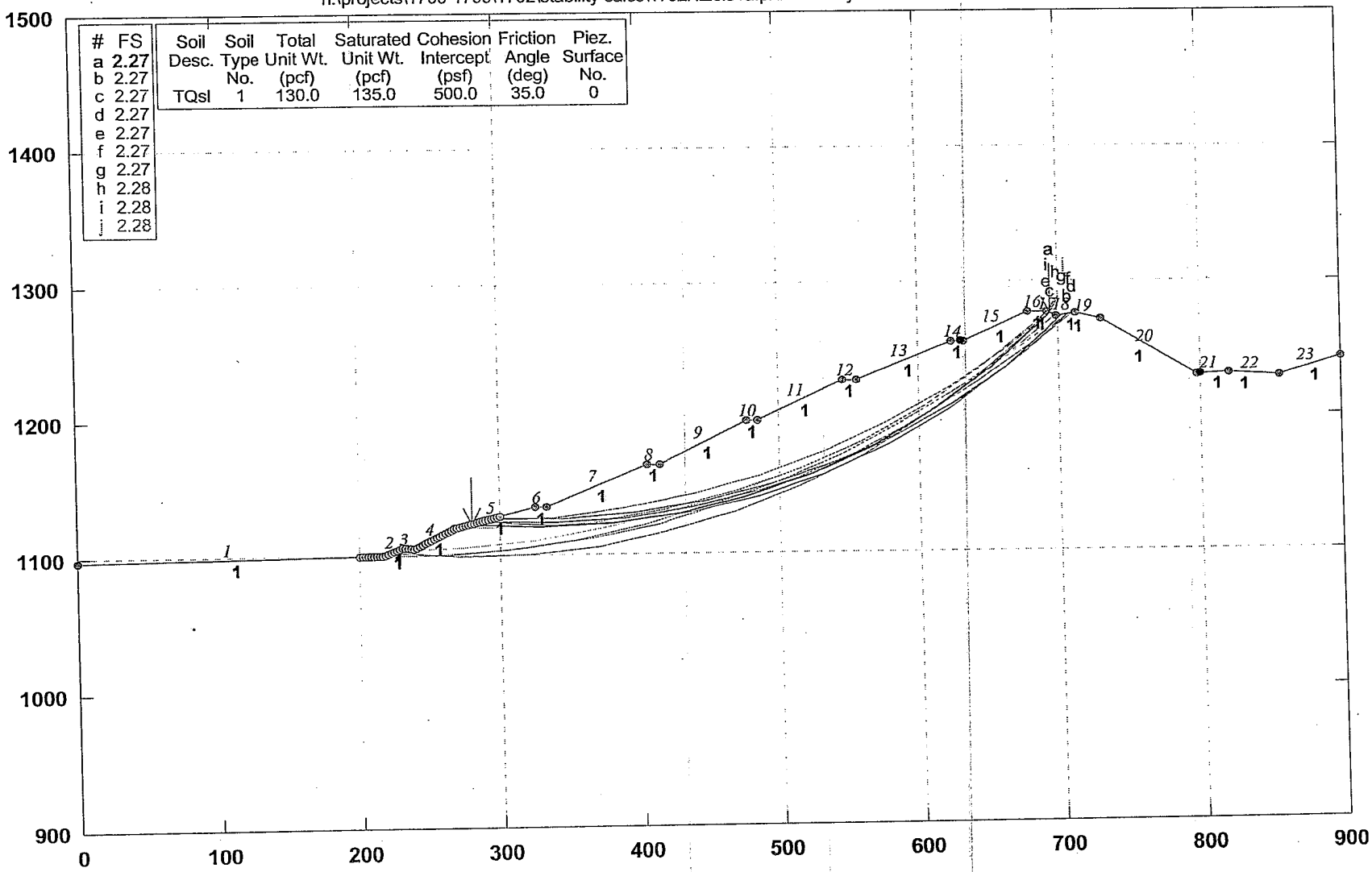
**** END OF GSTABL7 OUTPUT ****

Lndmrk off-site grading; 28L-28L'; CS-34L; circular; static

n:\projects\1700-1799\1702\stability calcs\1702r\t28ls1a.pl2 Run By: KPC 7/14/2008 02:57PM

#	FS
a	2.27
b	2.27
c	2.27
d	2.27
e	2.27
f	2.27
g	2.27
h	2.28
i	2.28
j	2.28

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
TQsl	1	130.0	135.0	500.0	35.0	0



GSTABL7 v.2 FSmin=2.27

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 29

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                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/14/2008
Time of Run:                02:57PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t28ls1a.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t28ls1a.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t28ls1a.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 28L-28L';
                      CS-34L; circular; static

BOUNDARY COORDINATES
   23 Top   Boundaries
   23 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
    1         0.00     1097.00      217.00     1100.00        1
    2       217.00     1100.00      231.00     1106.00        1
    3       231.00     1106.00      239.00     1105.00        1
    4       239.00     1105.00      268.00     1120.00        1
    5       268.00     1120.00      325.00     1135.00        1
    6       325.00     1135.00      333.00     1135.00        1
    7       333.00     1135.00      405.00     1167.00        1
    8       405.00     1167.00      414.00     1167.00        1
    9       414.00     1167.00      477.00     1198.00        1
   10       477.00     1198.00      485.00     1198.00        1
   11       485.00     1198.00      546.00     1227.00        1
   12       546.00     1227.00      556.00     1227.00        1
   13       556.00     1227.00      623.00     1256.00        1
   14       623.00     1256.00      632.00     1256.00        1
   15       632.00     1256.00      678.00     1277.00        1
   16       678.00     1277.00      692.00     1277.00        1
   17       692.00     1277.00      699.00     1274.00        1
   18       699.00     1274.00      712.00     1276.00        1
   19       712.00     1276.00      730.00     1271.00        1
   20       730.00     1271.00      798.00     1230.00        1
   21       798.00     1230.00      820.00     1231.00        1
   22       820.00     1231.00      856.00     1228.00        1
   23       856.00     1228.00      900.00     1242.00        1
User Specified Y-Origin =        900.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
 No.  (pcf)    (pcf)     (psf)     (deg)   Param.     (psf)     No.
  1   130.0    135.0     500.0     35.0    0.00       0.0       0
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
1500 Trial Surfaces Have Been Generated.
  30 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
Along The Ground Surface Between  X = 200.00(ft)
                              and  X = 300.00(ft)
Each Surface Terminates Between   X = 630.00(ft)
                              and  X = 800.00(ft)
```

Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
50.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -25.0
And 15.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 9
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.6 %
Statistical Data On All Valid FS Values:
FS Max = 3.703 FS Min = 2.265 FS Ave = 2.792
Standard Deviation = 0.307 Coefficient of Variation = 10.99 %
((Modified Bishop FS for Critical Surface = 2.266))
Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	279.59	1123.05
2	329.52	1120.32
3	379.47	1122.60
4	428.94	1129.86
5	477.43	1142.03
6	524.47	1158.99
7	569.57	1180.57
8	612.29	1206.55
9	652.20	1236.67
10	688.90	1270.63
11	693.83	1276.21

Circle Center At X = 331.78 ; Y = 1619.69 ; and Radius = 499.38
*** FOS = 2.265 Theta (ki=1.0) = 19.65 ***
Lambda = 0.357
Individual data on the 22 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	45.4	42597.0	0.0	0.0	0.	0.	0.0	0.0	0.0
2	4.5	8547.4	0.0	0.0	0.	0.	0.0	0.0	0.0
3	3.5	6610.2	0.0	0.0	0.	0.	0.0	0.0	0.0
4	46.5	143682.0	0.0	0.0	0.	0.	0.0	0.0	0.0
5	25.5	122336.2	0.0	0.0	0.	0.	0.0	0.0	0.0
6	9.0	46792.9	0.0	0.0	0.	0.	0.0	0.0	0.0
7	14.9	81376.3	0.0	0.0	0.	0.	0.0	0.0	0.0
8	48.1	314199.8	0.0	0.0	0.	0.	0.0	0.0	0.0
9	0.4	3142.0	0.0	0.0	0.	0.	0.0	0.0	0.0
10	7.6	53728.1	0.0	0.0	0.	0.	0.0	0.0	0.0
11	39.5	284797.8	0.0	0.0	0.	0.	0.0	0.0	0.0
12	21.5	161635.0	0.0	0.0	0.	0.	0.0	0.0	0.0
13	10.0	71913.2	0.0	0.0	0.	0.	0.0	0.0	0.0
14	13.6	92831.2	0.0	0.0	0.	0.	0.0	0.0	0.0
15	42.7	269700.2	0.0	0.0	0.	0.	0.0	0.0	0.0
16	10.7	59990.1	0.0	0.0	0.	0.	0.0	0.0	0.0
17	9.0	44429.7	0.0	0.0	0.	0.	0.0	0.0	0.0

18	20.2	82889.9	0.0	0.0	0.	0.	0.0	0.0	0.0
19	25.8	75474.8	0.0	0.0	0.	0.	0.0	0.0	0.0
20	10.9	16163.4	0.0	0.0	0.	0.	0.0	0.0	0.0
21	3.1	1861.6	0.0	0.0	0.	0.	0.0	0.0	0.0
22	1.8	340.3	0.0	0.0	0.	0.	0.0	0.0	0.0

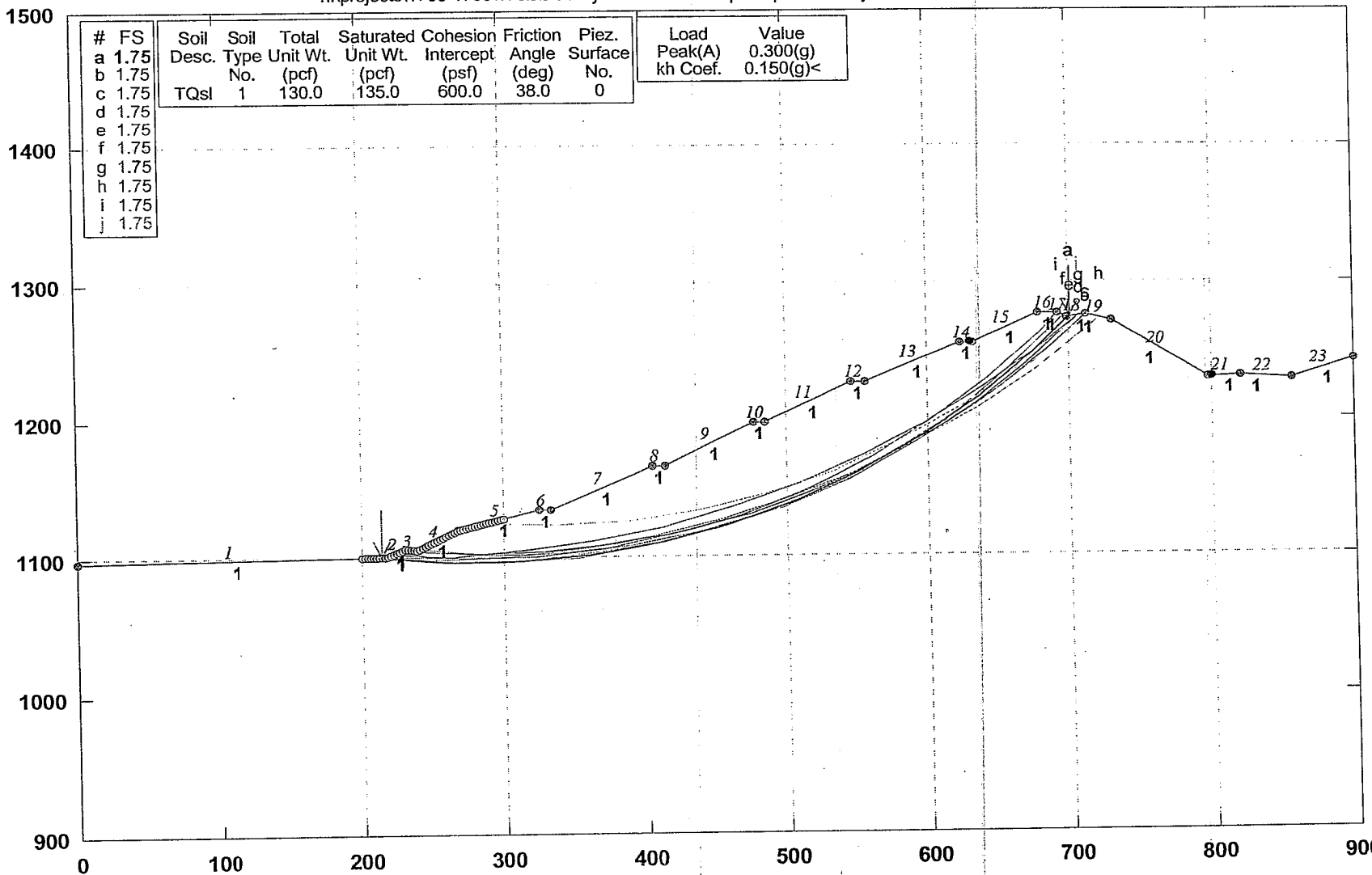
Lndmrk off-site grading; 28L-28L'; CS-34L; circular; pseudo

n:\projects\1700-1799\1702\stability calcs\1702r\t28lps1a.pl2 Run By: KPC 7/14/2008 02:58PM

#	FS
a	1.75
b	1.75
c	1.75
d	1.75
e	1.75
f	1.75
g	1.75
h	1.75
i	1.75
j	1.75

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
TQsl	1	130.0	135.0	600.0	38.0	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.75

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run No. 30

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                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
            (All Rights Reserved-Unauthorized Use Prohibited)
*******************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*******************************************************************************
Analysis Run Date:          7/14/2008
Time of Run:                02:58PM
Run By:                     KPC
Input Data Filename:        N:\Projects\1700-1799\1702\Stability Calcs\1702R\t28lpsla.in
Output Filename:            N:\Projects\1700-1799\1702\Stability Calcs\1702R\t28lpsla.OUT
Unit System:                English
Plotted Output Filename:    N:\Projects\1700-1799\1702\Stabi Calcs\1702R\t28lpsla.PLT
PROBLEM DESCRIPTION:  Lndmrk off-site grading; 28L-28L';
                      CS-34L; circular; pseudo

BOUNDARY COORDINATES
   23 Top   Boundaries
   23 Total Boundaries
Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.        (ft)        (ft)        (ft)        (ft)     Below Bnd
    1         0.00     1097.00      217.00     1100.00         1
    2       217.00     1100.00      231.00     1106.00         1
    3       231.00     1106.00      239.00     1105.00         1
    4       239.00     1105.00      268.00     1120.00         1
    5       268.00     1120.00      325.00     1135.00         1
    6       325.00     1135.00      333.00     1135.00         1
    7       333.00     1135.00      405.00     1167.00         1
    8       405.00     1167.00      414.00     1167.00         1
    9       414.00     1167.00      477.00     1198.00         1
   10       477.00     1198.00      485.00     1198.00         1
   11       485.00     1198.00      546.00     1227.00         1
   12       546.00     1227.00      556.00     1227.00         1
   13       556.00     1227.00      623.00     1256.00         1
   14       623.00     1256.00      632.00     1256.00         1
   15       632.00     1256.00      678.00     1277.00         1
   16       678.00     1277.00      692.00     1277.00         1
   17       692.00     1277.00      699.00     1274.00         1
   18       699.00     1274.00      712.00     1276.00         1
   19       712.00     1276.00      730.00     1271.00         1
   20       730.00     1271.00      798.00     1230.00         1
   21       798.00     1230.00      820.00     1231.00         1
   22       820.00     1231.00      856.00     1228.00         1
   23       856.00     1228.00      900.00     1242.00         1
 User Specified Y-Origin =        900.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
 Soil  Total  Saturated  Cohesion  Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1   130.0    135.0     600.0     38.0    0.00       0.0       0
 Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
 Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
 Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
 Specified Seismic Pore-Pressure Factor =   0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 1500 Trial Surfaces Have Been Generated.
   30 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
```

Along The Ground Surface Between X = 200.00(ft)
and X = 300.00(ft)
Each Surface Terminates Between X = 630.00(ft)
and X = 800.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
50.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -25.0
And 15.0 deg.

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 1500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 6
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 0.4 %
Statistical Data On All Valid FS Values:
FS Max = 2.721 FS Min = 1.745 FS Ave = 2.010
Standard Deviation = 0.141 Coefficient of Variation = 7.03 %
((Modified Bishop FS for Critical Surface = 1.764))
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	214.29	1099.96
2	264.14	1096.19
3	314.14	1096.73
4	363.91	1101.58
5	413.07	1110.69
6	461.26	1124.01
7	508.13	1141.43
8	553.32	1162.82
9	596.50	1188.03
10	637.35	1216.87
11	675.56	1249.12
12	700.59	1274.25

Circle Center At X = 283.10 ; Y = 1673.59 ; and Radius = 577.74
*** FOS = 1.745 Theta (ki=1.0) = 18.63 ***
Lambda = 0.337
Individual data on the 28 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Surcharge Ver (lbs)	Surcharge Load (lbs)
1	2.7	42.8	0.0	0.0	0.	0.	6.4	0.0	0.0
2	14.0	6865.4	0.0	0.0	0.	0.	1029.8	0.0	0.0
3	8.0	7388.4	0.0	0.0	0.	0.	1108.3	0.0	0.0
4	25.1	46938.3	0.0	0.0	0.	0.	7040.7	0.0	0.0
5	3.9	11426.4	0.0	0.0	0.	0.	1714.0	0.0	0.0
6	46.1	177484.5	0.0	0.0	0.	0.	26622.7	0.0	0.0
7	10.9	51263.2	0.0	0.0	0.	0.	7689.5	0.0	0.0
8	8.0	38295.7	0.0	0.0	0.	0.	5744.3	0.0	0.0
9	30.9	167925.2	0.0	0.0	0.	0.	25188.8	0.0	0.0
10	41.1	280379.5	0.0	0.0	0.	0.	42056.9	0.0	0.0
11	8.1	59837.9	0.0	0.0	0.	0.	8975.7	0.0	0.0
12	0.9	6812.6	0.0	0.0	0.	0.	1021.9	0.0	0.0

13	47.3	375697.3	0.0	0.0	0.	0.	56354.6	0.0	0.0
14	15.7	137483.7	0.0	0.0	0.	0.	20622.5	0.0	0.0
15	8.0	69320.9	0.0	0.0	0.	0.	10398.1	0.0	0.0
16	23.1	199542.8	0.0	0.0	0.	0.	29931.4	0.0	0.0
17	37.9	332839.1	0.0	0.0	0.	0.	49925.9	0.0	0.0
18	7.3	62717.4	0.0	0.0	0.	0.	9407.6	0.0	0.0
19	2.7	22088.6	0.0	0.0	0.	0.	3313.3	0.0	0.0
20	40.5	313555.8	0.0	0.0	0.	0.	47033.4	0.0	0.0
21	26.5	182169.5	0.0	0.0	0.	0.	27325.4	0.0	0.0
22	9.0	53918.1	0.0	0.0	0.	0.	8087.7	0.0	0.0
23	5.3	29363.0	0.0	0.0	0.	0.	4404.4	0.0	0.0
24	38.2	169743.5	0.0	0.0	0.	0.	25461.5	0.0	0.0
25	2.4	8286.2	0.0	0.0	0.	0.	1242.9	0.0	0.0
26	14.0	33501.2	0.0	0.0	0.	0.	5025.2	0.0	0.0
27	7.0	5795.3	0.0	0.0	0.	0.	869.3	0.0	0.0
28	1.6	140.5	0.0	0.0	0.	0.	21.1	0.0	0.0

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EVALUATION OF POTENTIAL FOR LIQUEFACTION, EARTHQUAKE-INDUCED SETTLEMENT, AND LATERAL SPREADING

1. Introduction

The potential for liquefaction, earthquake-induced settlement, and lateral spreading at the Landmark Village site were previously assessed and presented in our Geologic and Geotechnical Report - Vesting Tentative Tract 53108, dated 9/27/00. We have subsequently updated our liquefaction assessment in conformance with the current criteria of the State of California and reassessed our recommendation for removal depths and potential grading constraints.

Potentials for liquefaction and for earthquake-induced settlement were reassessed (in accordance with current criteria of the County of Los Angeles and the DMG SP117 Guidelines referenced in those criteria) for 21 of the 64 CPT soundings that were previously performed at the site. The procedures presented in the following references were used for this evaluation:

- *Liquefaction Resistance of Soils: Summary Report* (NCEER/NSF, 2001), by Youd, Idriss, et al.

- *Cyclic Liquefaction and its Evaluation Based on the SPT and CPT,* by Robertson and Wride, 1997.

- *Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement,* Youd, Hanson, Corbett and Bartlett, ASCE Journal of Geotechnical and Geoenvironmental Engineering, December 2002.

- *Evaluation of Settlements In Sands Due To Earthquake Shaking,* Tokimatsu and Seed, ASCE Journal of Geotechnical Engineering, August, 1987.

Field and laboratory data obtained during previous investigations were used in our analyses.

The liquefaction potential and seismic settlement analyses were performed in accordance with current criteria of the County of Los Angeles and the DMG SP117 Guidelines referenced in those criteria and updated in conformance with the requirements of the

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2007 edition of the California Building Code (CBC). Our analyses indicate that potential liquefaction-induced settlement will be less than one inch at 11 of the 21 CPT sounding locations that we analyzed and that potential liquefaction-induced settlement between about one and two inches could occur in the vicinity of CPT soundings 5, 13, 19, 96, 105, 106, 108, 109, 111, and 115. We have made minor revisions to the recommended alluvial removal depths shown on the attached Geotechnical Map in order to mitigate potential liquefaction-induced settlements in the vicinity of these CPT soundings.

Potential for lateral spreading at the project site is believed to be low since laterally continuous, potentially liquefiable soil layers with a relative density corresponding to $N1_{60} \leq 15$ are not present at the site.

2. Data and Assumptions

Factors that affect potential for liquefaction and seismic settlements at the project site include estimated ground motion parameters, engineering characteristics of site soils, and groundwater depth.

a. Estimated Ground Motion Parameters

In compliance with new criteria presented in the 2007 CBC, an updated Probabilistic Seismic Hazard Analysis (PSHA) was completed to evaluate the design basis ground acceleration at the site for use in our liquefaction analysis. The computer program FRISKSP (version 4.0) by Thomas Blake was used in the analysis. Per CBC criteria, we assessed potential accelerations with a 2% chance of exceedance in a 50 year period, i.e. a 2475 year return period. Per CBC criteria, two-thirds of this acceleration should be used for analysis.

The unweighted acceleration with a 2% chance of exceedance in 50 years at the site is estimated to be 1.24g, and two-thirds of this value is 0.83g. The most likely scenario generating this acceleration is a 6.5 magnitude earthquake on the Santa Susana fault. This design acceleration value is nearly the same as, but slightly lower than, the acceleration value of 0.87g with a 10% chance of exceedance in 50 years estimated during our previous analyses of the site. The acceleration weighted for a standardized 7.5 magnitude earthquake is 0.56g, which is also slightly lower than the 0.59g value estimated in our previous analyses. The reported accelerations are

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the unweighted average of the three accelerations indicated by the attention relationships of Boore et al (1997) for site Class D, Sadigh et al (1997) for deep soil, and Campbell and Bozorgnia (1997) for alluvium (see summary **Table C2** and sample output file for Boore et al (2007) run for details). Weighted and unweighted accelerations of 0.56g and 0.83g, respectively, were used in our updated liquefaction analyses.

It should be emphasized that the ground acceleration values presented in our report are based on simplified curves of fault rupture area to magnitude, and ground motion attenuation relationship which represent averages of highly variable data measured during historic earthquakes. Predicted accelerations should be considered rough estimates rather than precise facts and, therefore, ground accelerations at the subject site from future seismic events may exceed the predicted accelerations. Due to the dip-slip nature of most of the faults in southern California, vertical accelerations may equal horizontal accelerations. Ground motions may originate from virtually any direction due to the presence of major faults in all directions from the site.

b. Engineering Characteristics of Site Soils

Engineering characteristics of site soils were interpreted principally from data from Cone Penetration Test (CPT) soundings. These data include tip resistance (q_c), friction ratio (f_s), interpreted soil types and fines contents, and groundwater levels from dynamic groundwater pressure dissipation tests. As noted in our referenced 9/27/00 report, there is substantial consistency between the soil types and the SPT blow counts observed in rotary-wash borings RW-11 and RW-13 and the soil types and SPT blow counts interpreted at adjacent locations in CPT soundings CPT-103 and CPT-109, respectively.

c. Ground Water Depth

Existing ground water depths and Historic High ground water depths at the locations of the CPT soundings are provided in **Table C1**.

Historic High Ground Water depth was adopted herein for evaluation of liquefaction potential and associated phenomena.

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In-situ ground water depths were used for correcting SPT blow count data measured during sampling of boreholes to the standard effective overburden pressure (1.0 tsf) used in liquefaction analyses. Effective vertical soil pressure based on the historic high ground water depth was used to estimate cyclic shear stress induced in site soils by earthquake shaking.

3. CPT Logs and Interpretations

Two versions of CPT logs and soil type interpretations have been prepared for each CPT sounding. Soil type interpretation in the first version of the CPT logs, which were prepared by the company that performed the CPT soundings, is based on the procedures described by Robertson and Campanella, 1989. This version is presented in our referenced 9/27/00 report. The interpreted soil types from this version are used herein for estimation of equivalent SPT blow count values (for comparison with SPT blow count data from exploration borings).

The second version of the CPT logs was prepared by AESEGI. The interpreted soil types and percent fines provided in these logs were evaluated based on the procedures presented in *Liquefaction Resistance of Soils: Summary Report* (NCEER/NSF, 2001), by Youd, Idriss, et al. Measured cone-tip resistance (q_c), friction ratio (f_s), and interpreted corrected SPT resistance ($N1_{60}$) from the first version of the CPT logs were used to evaluate the interpreted soil types and normalized q_c and f_s values shown on the second version of the CPT logs. This version is presented as an attachment at the end of this appendix.

4. Methods of Analysis

a. Assessment of Liquefaction Potential

Liquefaction is a phenomenon in which porewater pressure generated by earthquake shaking causes sudden, temporary loss of shear strength in saturated, typically granular soils. Foundations founded on liquefied soils may settle and/or move laterally.

Liquefaction potential analyses generally are performed by a method first proposed by Seed and Idriss (1970) and subsequently revised by Youd, Idriss, et al (NCEER/NSF, 2001). In these methods, earthquake-induced cyclic shear stresses in

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the ground are estimated and compared with empirically based cyclic shear strength (resistance) derived from data from case histories in which liquefaction was observed and case histories in which liquefaction was not observed.

Estimation of cyclic shear strength against liquefaction is based on measured CPT tip resistance (q_c) and friction ratio (f_s), percent fines estimated from CPT data, and estimated in-situ effective overburden pressure. CPT tip penetration resistance data corrected to 1.0 tsf pressure (q_{c1N}) are used to calculate the Cyclic Resistance Ratio (CRR) for clean sandy soils subjected Magnitude 7.5 earthquakes. The overburden pressure correction for the CPT tip resistance is calculated using the following equations from Youd, Idriss, et al (NCEER/NSF, 2001):

$$C_Q = (1/\sigma'_o)^{0.5}, \quad 0.5 \leq C_Q \leq 1.7$$
$$q_{c1N} = C_Q \cdot q_c$$

where: σ'_o = effective overburden pressure [tsf or kg/cm^2]
q_c = uncorrected CPT tip resistance [tsf or kg/cm^2]

The overburden pressure corrected CPT tip resistance (q_{c1N}) is further corrected for fines content in order to obtain an equivalent clean sand CPT tip resistance value, as recommended by Youd, Idriss, et al (NCEER/NSF, 2001) and by Robertson & Wride (NCEER, 1997). The fines content corrected CPT tip resistance, $(q_{c1N})_{cs}$, is used to calculate the Cyclic Resistance Ratio (CRR) against liquefaction, in accordance with Youd, Idriss, et al (NCEER/NSF, 2001).

The earthquake-induced cyclic shear stress ratio (CSR or τ_{av}/σ'_v) for each depth in question is calculated using the following equation by Youd, Idriss, et al (NCEER/NSF, 2001):

$$\tau_{av}/\sigma'_v = 0.65 \cdot (a_{max}/g) \cdot \sigma_o/\sigma'_v \cdot r_d$$

Where: a_{max}/g = Peak horizontal ground surface acceleration (PGHA) divided by the acceleration of gravity
σ_o = Total overburden pressure at the depth in question
σ'_v = Effective overburden pressure at the depth in question
r_d = Shear stress reduction factor which accounts for soil deformability at the depth in question

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Cyclic Resistance Ratio (CRR) vs. corrected cone-tip resistance $(q_{c1N})_{cs}$ curves are presented by Youd, Idriss, et al (NCEER/NSF, 2001). These curves are based on case histories in which liquefaction was observed / not observed in clean sands subjected to earthquakes with a magnitude of about 7.5. Soils with $(q_{c1N})_{cs}$ and CSR values that plot above these curves are potentially liquefiable and soils with $(q_{c1N})_{cs}$ and CSR values that plot below these curves generally are not liquefiable. In accordance with the requirements of the County of Los Angeles Department of Public Works, a soil deposit is considered to be liquefiable if its factor of safety against liquefaction (i.e., CRR divided by CSR) is less than 1.3.

Values of the factor of safety against liquefaction for the soil profiles in each of the CPT logs were calculated using fines content corrected values of tip resistance (q_{c1N}) at 6-inch depth intervals, per Youd, Idriss, et al (NCEER/NSF, 2001). A typical spreadsheet that illustrates the factor of safety calculations is shown on **Figure C22** – Typical Spreadsheet of Liquefaction Potential Analyses. Non-liquefiable soil deposits may settle as a result of earthquake shaking. This non-liquefied, earthquake-induced settlement is included in our settlement estimates, as discussed in the following section.

b. Estimation of Earthquake-Induced Settlements

Tokimatsu and Seed (1987) provide procedures for estimation of earthquake-induced settlement of unsaturated sands and of saturated sands. These procedures were used herein to estimate the distribution of potential earthquake-induced settlement vs. depth. The Tokimatsu and Seed method for estimation of earthquake-induced settlement is described below.

One-dimensional volumetric strain of unsaturated sandy soils caused by earthquake shaking is estimated based on estimated cyclic shear strain and $N1_{60}$ values (estimated from the CPT soundings). This one-dimensional strain is doubled in order to estimate strain produced by multi-directional earthquake shaking. Settlement is calculated by multiplying the volumetric strain value by the thickness of the soil layer. Although this procedure for estimating earthquake-induced settlement was developed based on data from clean to slightly silty sands, CPT tip resistances (q_c) used herein to estimate $N1_{60}$ values of unsaturated soils were not

corrected for fines content. Omitting the fines content correction yields conservative (i.e., higher) settlement estimates for silty sands and silts.

Volumetric strain of saturated sands caused by earthquake shaking is estimated using earthquake-induced cyclic shear stress (τ_{av}) and $N1_{60}$ values derived from CPT tip resistance (q_c) values. Since the design chart for volumetric strain of saturated sands includes the effect of multi-directional earthquake shaking, it is not necessary to double the volumetric strain calculated for saturated sand deposits. Although the referenced procedure strictly applies only to saturated clean sands, it may also be used to estimate settlement in silty sands because the interpreted $N1_{60}$ values used to estimate volumetric strain are based on q_c values which have been corrected for fines content (i.e., an equivalent CPT tip resistance that corresponds to clean sand is used to estimate $N1_{60}$ for silty sand and silt deposits). Settlement is calculated by multiplying the thickness of the saturated soil layer by the volumetric strain.

c. Estimation of Potential For Lateral Spreading

Regression equations for estimation of potential magnitude of lateral spread displacement that can occur if liquefaction occurs in soils beneath flat to gently sloping sites were developed by Youd, Hanson, Corbett, and Bartlett (ASCE, December 2002). According to these equations, the magnitude of lateral spreading displacement depends on earthquake magnitude, site to fault distance, slope of the site surface, thickness of potentially liquefiable deposits at the site with an $N1_{60} \leq 15$ (T_{15}), and the median grain size and percent fines in the T_{15} layer. The data base used to develop the regression equations was limited to T_{15} values between 1 and 15 meters. It may be inferred from this that significant lateral spreading was not observed by the researchers at sites with less than about 1 meter of potentially liquefiable soils with $N1_{60} \leq 15$ blows/foot. Accordingly, it is assumed herein that earthquake shaking will not cause lateral spreading at a site unless it is underlain by a liquefiable layer that is at least about 1 meter in total thickness and which has an $N1_{60} \leq 15$ blows/foot.

In addition, the data base used to develop the regression equations for the magnitude of lateral spread displacement is based on case histories in which the liquefiable (T_{15}) layers extended laterally for substantial distances without being impeded by boundary effects. It may be inferred that significant lateral spreading has not been

observed at narrow sites that are bounded by denser soils or bedrock, or at sites where liquefiable (T_{15}) layers are laterally discontinuous. Accordingly, it is assumed herein that earthquake shaking will not cause lateral spreading at a site that does not have a laterally continuous, liquefiable layer with $N1_{60} \leq 15$ blows/foot.

5. Results of Analyses

a. Liquefaction Potential Assessment

Results of our liquefaction analyses using data from the selected CPT soundings are graphically summarized herein on individual sheets (Analysis of Liquefaction Potential Based q_c and f_s from CPT Data - **Figures C1 through C21**). These sheets present plots of depth vs. normalized CPT tip resistance and friction ratio, fines content interpreted from the CPT data, cyclic shear stress induced by earthquake shaking, maximum resistance to cyclic shear stress interpreted from the CPT data, and cumulative settlement that would be caused by earthquake shaking. The sheets also display groundwater depth at the time of the CPT soundings and estimated historic high groundwater depths.

b. Earthquake-Induced Settlement

Potential earthquake-induced settlement that could result from liquefaction prior to proposed grading was calculated (per CDMG Special Publication 117, 1997) to vary from 0 to about 1.9 inches at the locations of 21 CPT soundings (see Table C1). At the locations of CPT soundings 5, 13, 19, 96, 105, 106, 108, 111, and 115, potential liquefaction-induced settlements range from 1.0 to about 1.9 inches. Potential liquefaction-induced *differential* settlements at the locations of these CPT soundings varies from about 0.6 to 1.2 inches. Options to mitigate these potential differential settlements are provided below.

c. Lateral Spreading

As stated above, significant lateral spreading is expected only if a site is underlain by a laterally continuous, liquefiable layer that is at least about 1 meter thick and that has a relative density that corresponds to an $N1_{60}$ value of about 15 blows/ft, or less. Potential for lateral spreading due to liquefaction at the project site is low

because there is no laterally continuous, potentially liquefiable layer with a relative density that corresponds to $N1_{60} \leq 15$ blows/foot.

6. Conclusions

Our analyses indicate that potential liquefaction-induced settlement will be less than one inch at 11 of the 21 CPT sounding locations that we analyzed and that potential liquefaction-induced settlement between about one and two inches could occur in the vicinity of CPT soundings 5, 13, 19, 96, 105, 106, 108, 109, 111, and 115. We have made minor revisions to the recommended alluvial removal depths shown on the attached Geotechnical Map in order to mitigate potential liquefaction-induced settlements in the vicinity of these CPT soundings. Of the ten CPT sounding locations where potential liquefaction-induced settlements exceed one inch, seven can be mitigated with standard grading (removal and recompaction) techniques without dewatering. At two closely spaced locations (CPT soundings 105 and 106) removals to reduce to the seismic settlements to less than one inch would extend up to 4.5 ft below the observed ground water depth. Considering the granular nature of the soils, the shallow intrusion into the water table, and proposed deeper removals and dewatering for the adjacent channel liner, it is anticipated that the recommended removals will be feasible by means of grading and minor dewatering with slot trenches.

At the location of CPT-13, a removal depth of 23 ft is indicated to limit total seismic settlements to less than 1 in. Ground water depth was not accurately measured at this CPT because the sounding hole caved at 12 ft depth. For purposes of analysis, we assumed existing ground water depth to be 15 ft. However, subsequent measurements from the nearby CPT-96 and Piezometer P-32F indicate that the depth of existing ground water is probably at least 20 ft. deep in the vicinity of CPT-13. Therefore, the recommended removals probably extend only 3 ft below the existing water table. In light of the limited extent of recommended removals below the water table at this location, it is anticipated that shallow dewatering with slot trenches will be sufficient for excavation and dewatering of the nearby channel liner. If the recommended removals below the water table cannot be performed, buildings in the vicinity of CPT-13 should be designed for 1.2 in of total seismic settlement in addition to static settlement.

In summary, if the recommended removals are performed total seismic settlements will be less than 1 in and the differential seismic settlements will be 0.67 in or less. Following the grading, static differential settlements are anticipated to be 0.25 in or less.

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Therefore, total differential settlement between adjacent footing foundations is anticipated to be less than 1 in.

Potential for lateral spreading at the project site is low because there is no laterally continuous, potentially liquefiable layer with a relative density that corresponds to $N1_{60} \leq 15$ blows/foot.

The following attachments complete this Appendix.

- Liquefaction Potential Assessment References
- Summary Table of Subsurface Data, Potential Earthquake-Induced Settlements, and Removal Depths — Table C1
- Graphs of Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data) — Figures C1 thru C21
- Typical Spreadsheet of Liquefaction Potential Analyses — Figure C22
- AESEGI Interpreted CPT Data
- Output File for Boore et al (1997) FRISKSP Run
- Summary of Calculated Peak Horizontal Ground Accelerations — Table C2

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APPENDIX C

Analyzed Location CPT	Potential Earthquake-Induced Settlement (Inches)		Proposed Additional Fill (feet)	Recommended Removals (feet)	Total Compacted Fill Cap Thickness (feet)	Thickness of Non-Liquefiable soils below proposed grades (feet)	Total Thickness of Potentially Liquefiable Soils Following Removals (feet)	Ground water Table Depth (feet)	
	Before Removals	After Removals						Existing GWT	Historic GWT
2	0.91	0.88	6	5	11	19	5	28	10
5	1.49	0.82	3.5 cut	14 [1]	10.5	9.5	6.5	24	11
7	0.48	0.48	7	5	12	24	3.5	20	17
13	1.58	0.98	3.5	23	19.5	17.5	7	15 [2]	12
15	0.43	0.42	1.5	4	5.5	14.5	3.5	27	16
19	1.04	1.00	12	5	17	25	7.5	15	9
96	1.45	0.95	4	22	26	25.5	6	22.2	15
101	0.37	0.37	18	4	22	27.5	3.5	9.3	4
103	0.66	0.58	29	8	37	37	5	9.5	3
105	1.35	0.74	20	9	29	29	4.5	7.4	3
106	1.49	0.99	20	11	31	34	4	7.5	2
108	1.14	1.00	1	13	14	19	5.5	25.1	16
109	1.11	0.91	7	16	23	22	6.5	25	15
110	0.76	0.75	10.5	5	15.5	22.5	4.5	24	12
111	1.23	0.97	8.5	13	21.5	21.5	6	21.9	11
114	0.64	0.62	11.5	5	16.5	23.5	3	22.4	12
115	1.84	0.94	10	16	26	40	6.5	21.5	11
118	0.50	0.50	16	4	20	25	4	10.5	4

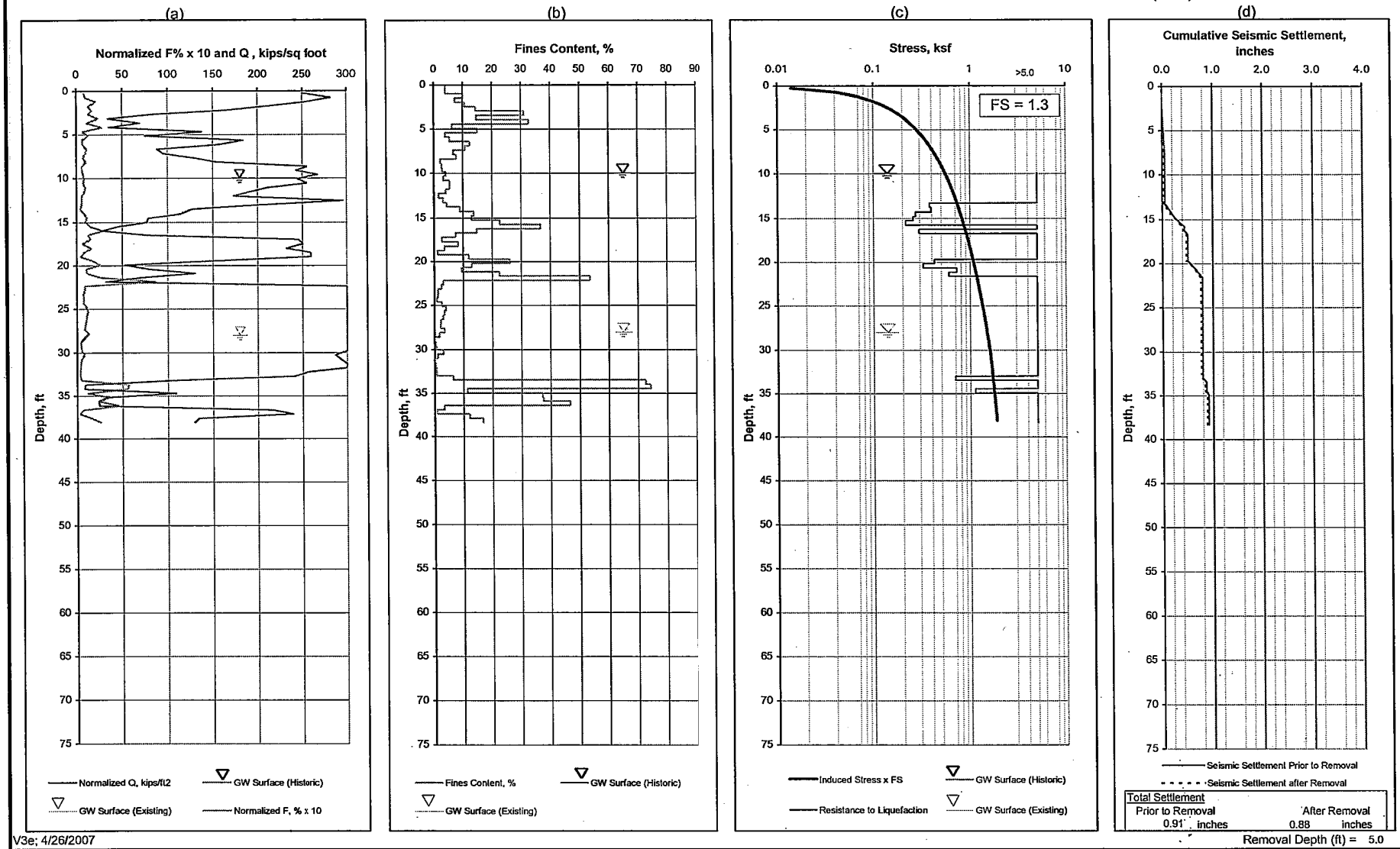
[1] Indicates removal depth in River Bank Slope Protection zone. In this area, excavation for placement of the typical Slope Protection system would be substantially deeper than the removal depth for mitigating potential seismic structural settlement at the respective CPT-5 location indicated in the table.

[2] A ground water depth of 15 ft was conservatively estimated. Subsequent data indicated that the existing ground water depth is probably at least 20 ft.

APPENDIX C

Analyzed Location CPT	Potential Earthquake-Induced Settlement (Inches)		Proposed Additional Fill (feet)	Recommended Removals (feet)	Total Compacted Fill Cap Thickness (feet)	Thickness of Non-Liquefiable soils below proposed grades (feet)	Total Thickness of Potentially Liquefiable Soils Following Removals (feet)	Ground water Table Depth (feet)	
	Before Removals	After Removals						Existing GWT	Historic GWT
122A	0.73	0.73	29	4	33	34.5	6.5	4	4
125	0.00	0.00	20	4	24	32	<1	11	9
128	0.28	0.24	0	9	9	19	1.5	26	20

Location..... **CPT-02**
Elevation (MSL)..... 964 ft



V3e; 4/26/2007

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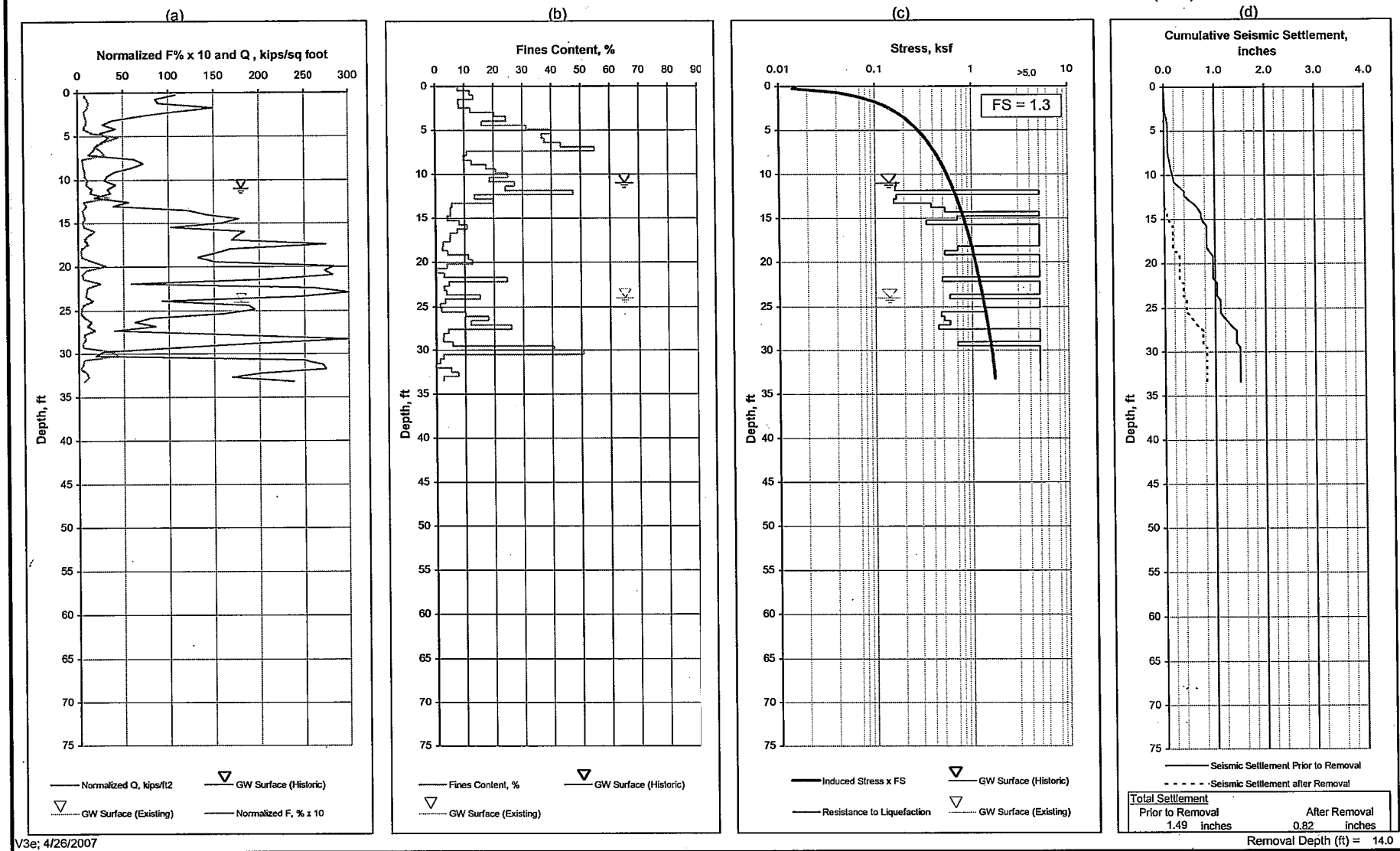
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C1
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-05
Elevation (MSL)..... 954 ft



V3e; 4/26/2007

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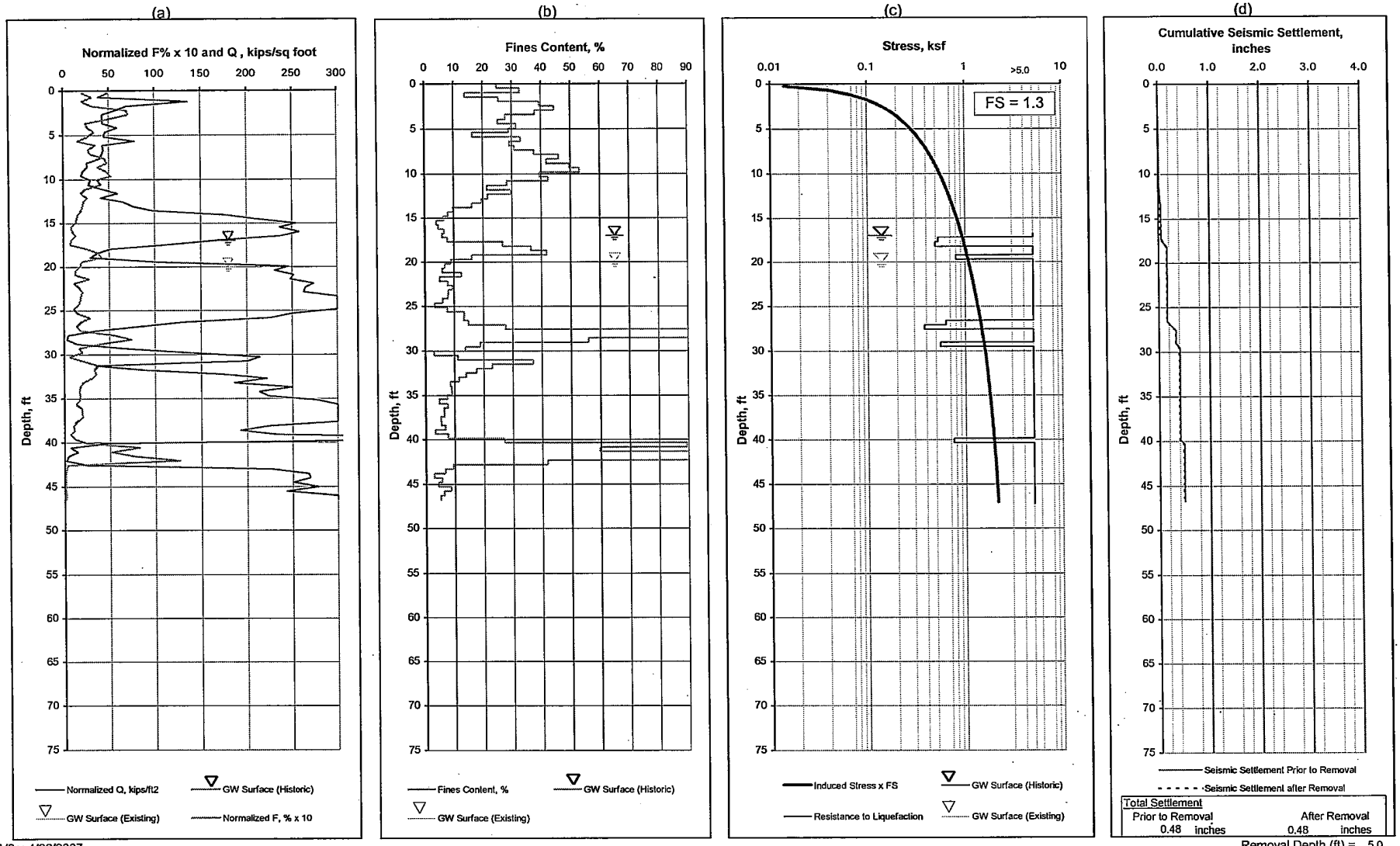
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C2
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-07**
Elevation (MSL)..... 955 ft



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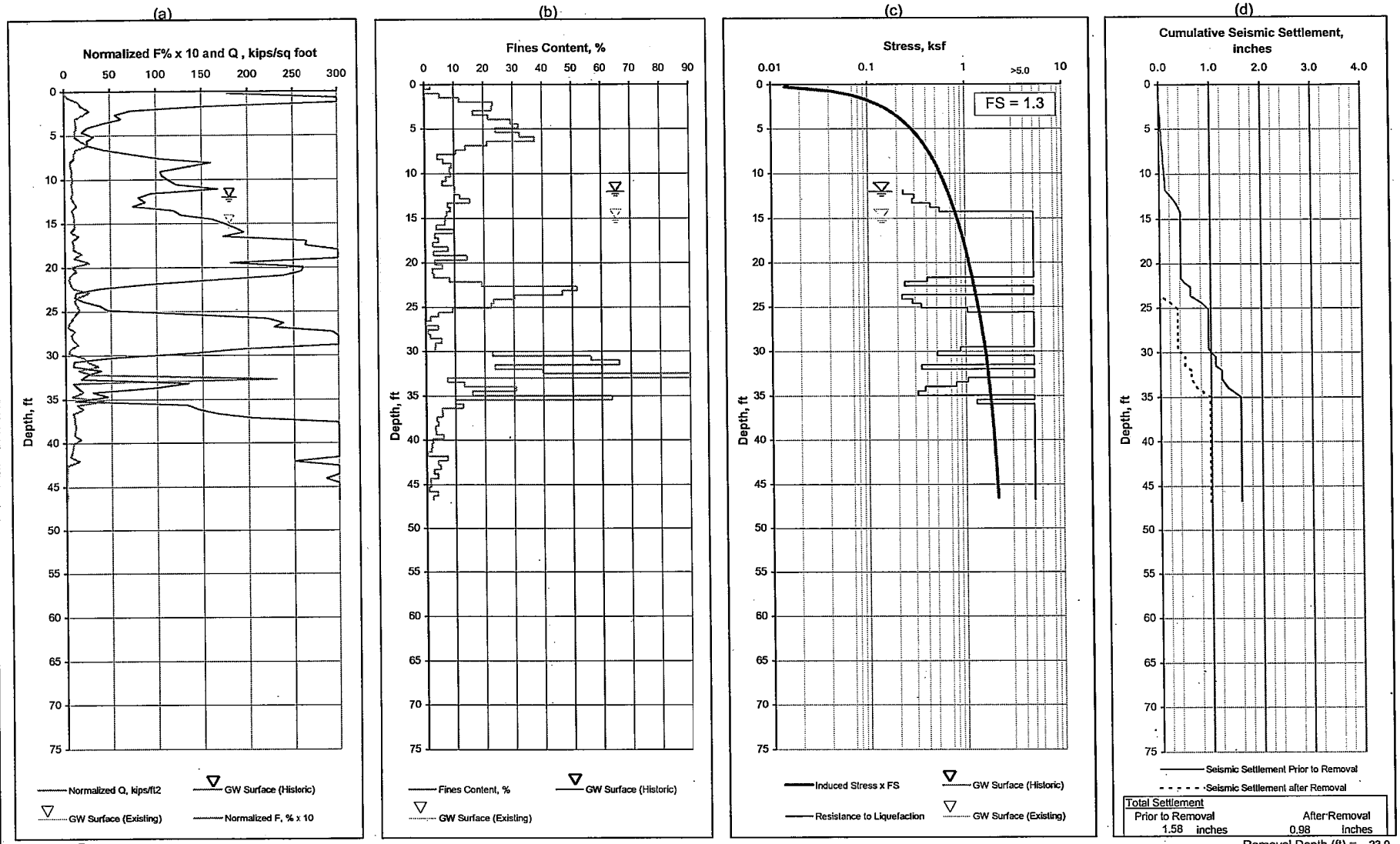
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No..	C3
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-13
Elevation (MSL)..... 940 ft



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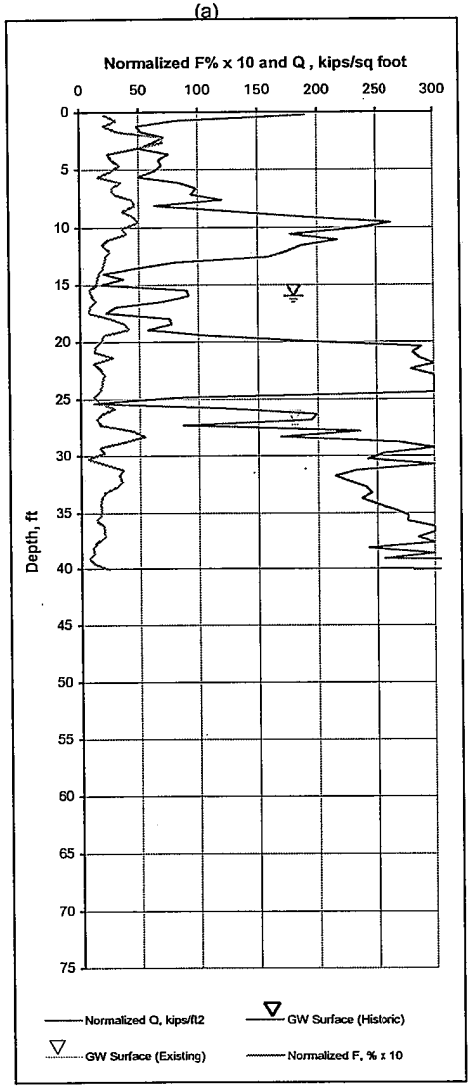
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

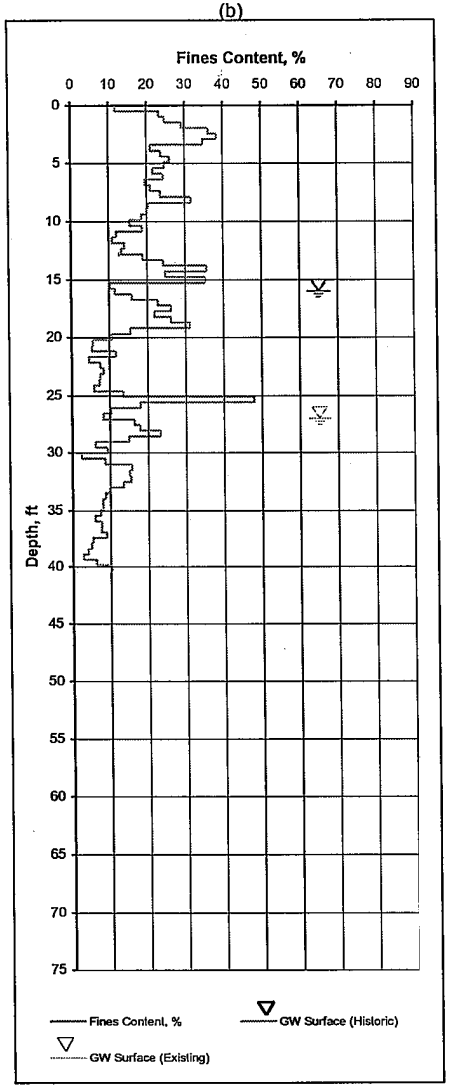
Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C4
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-15
Elevation (MSL)..... 946 ft

(a)



(b)



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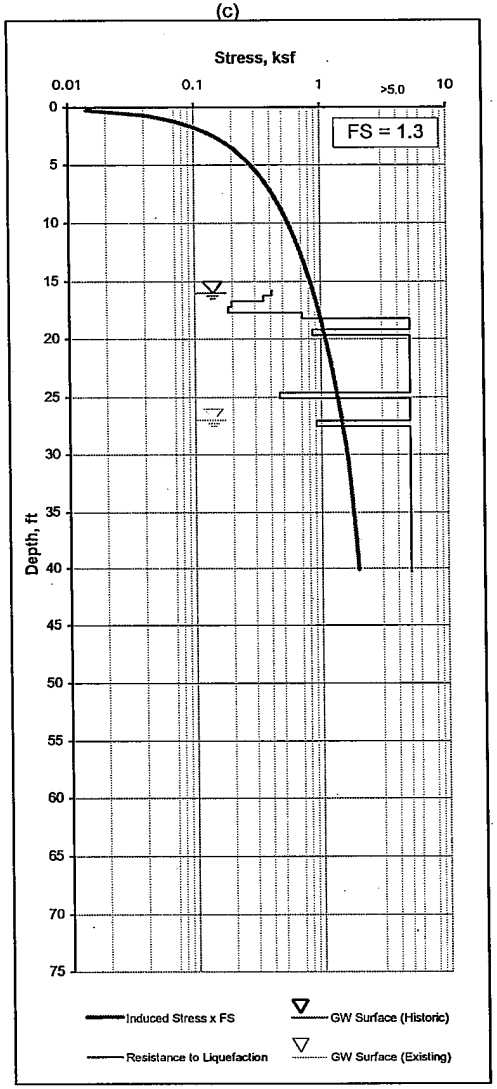

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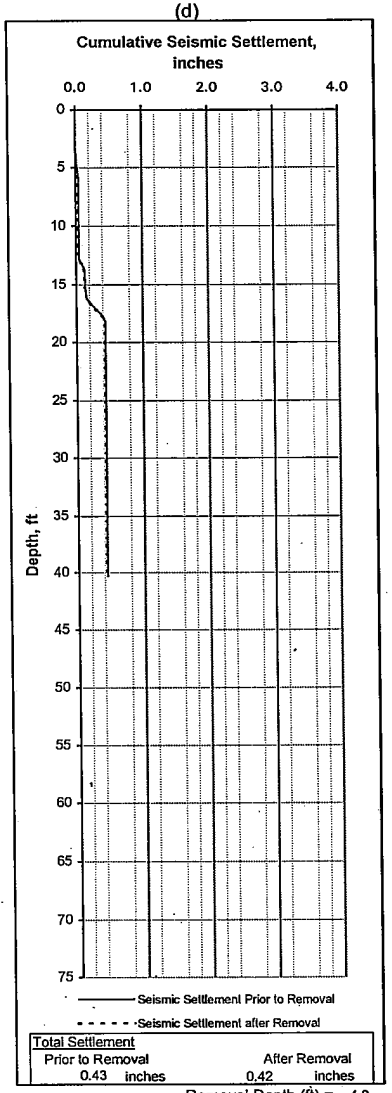

Removal Depth (ft) = 4.0

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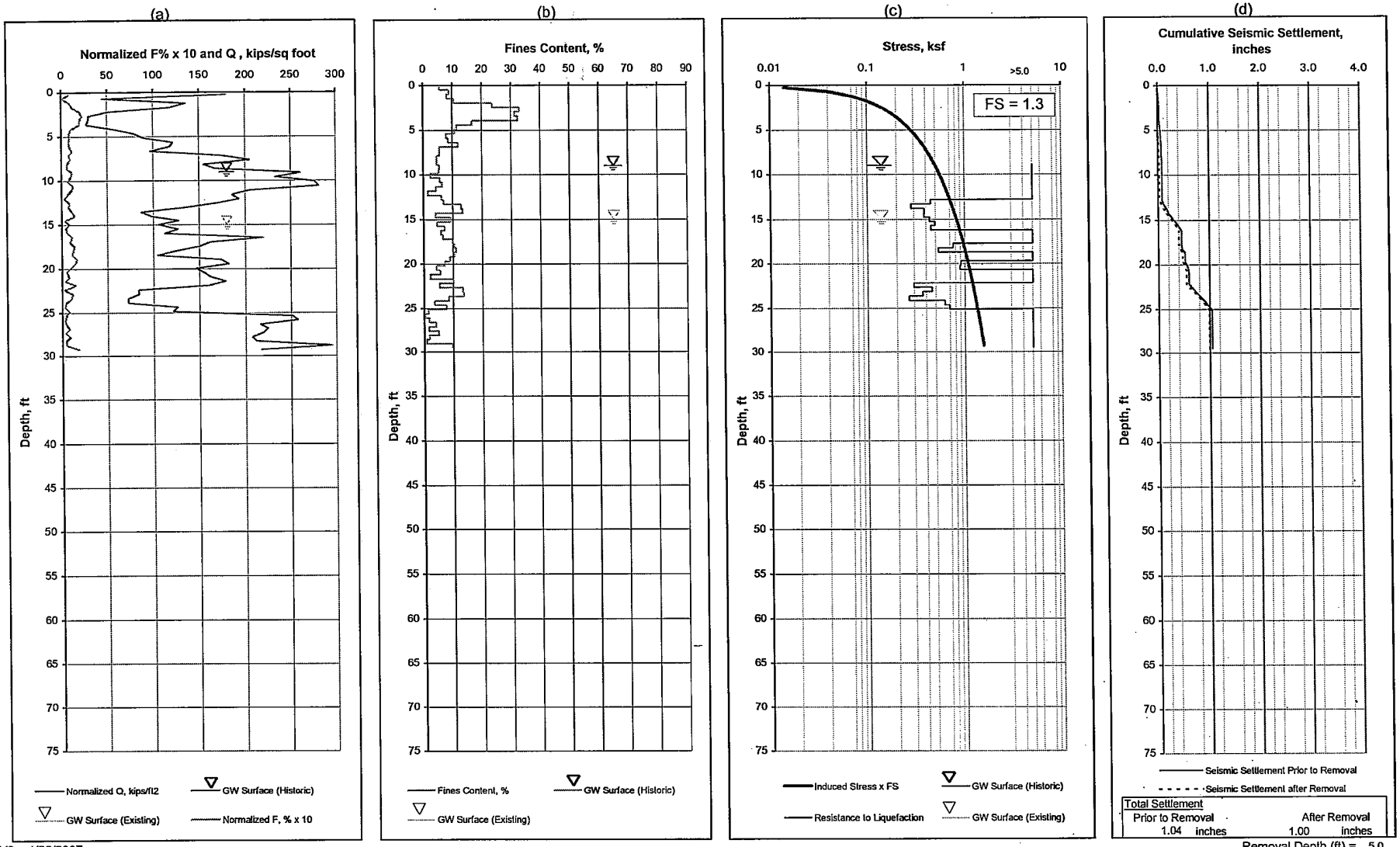
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C5
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-19
Elevation (MSL)..... 923 ft



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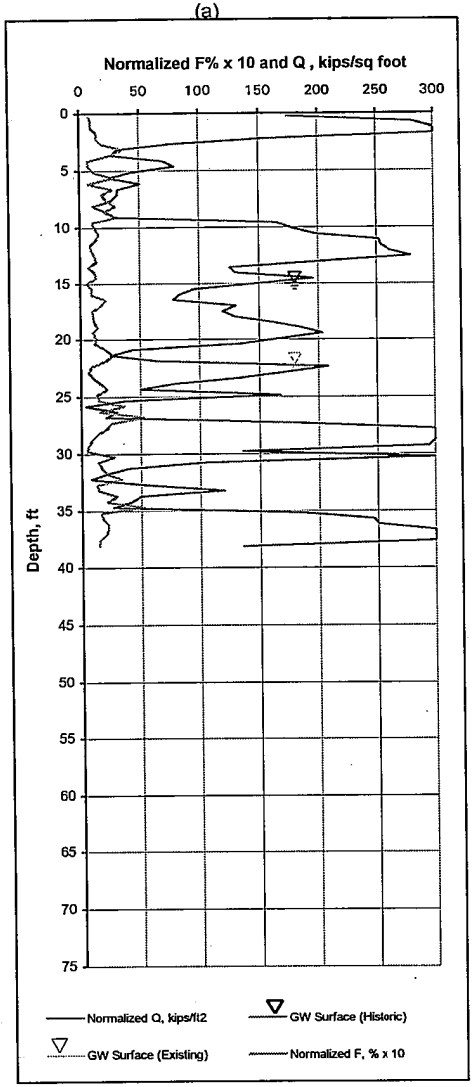
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C6
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-96
Elevation (MSL)..... 964 ft

(a)



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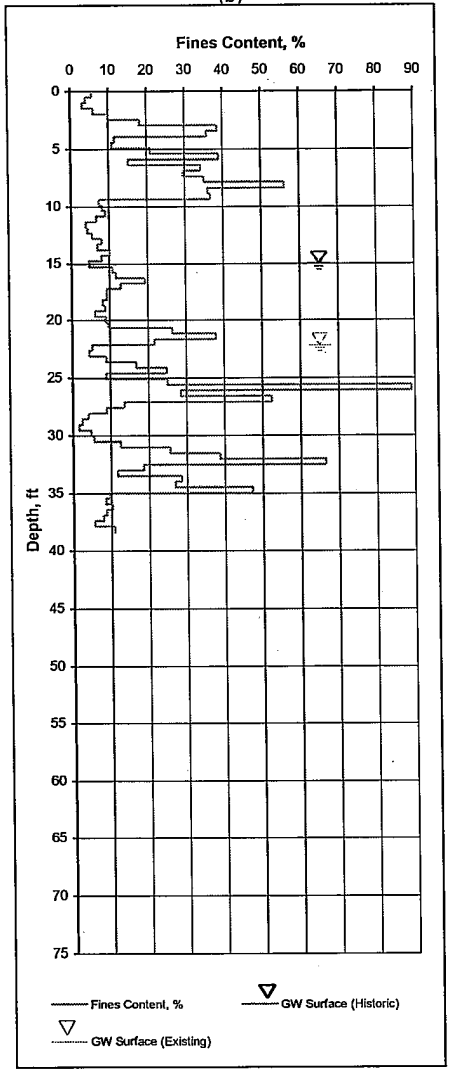

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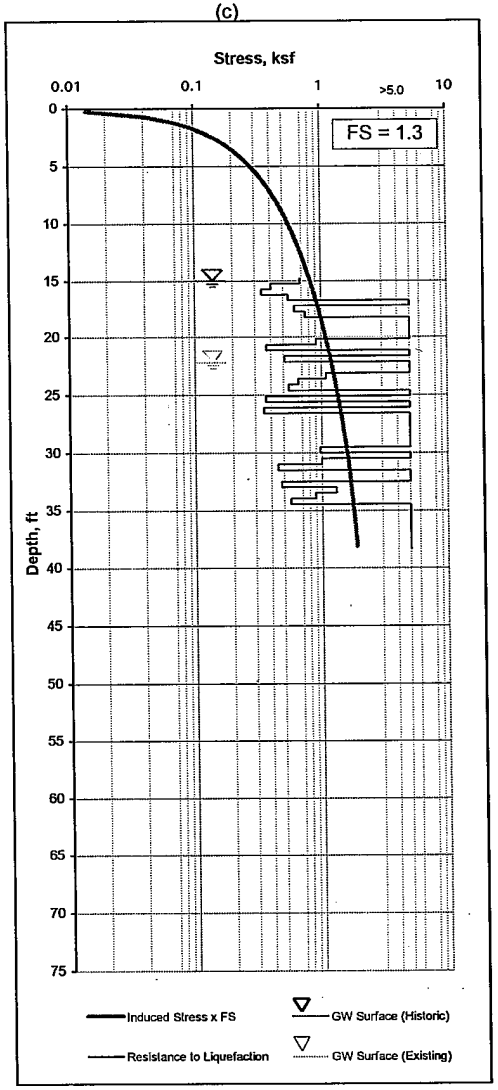

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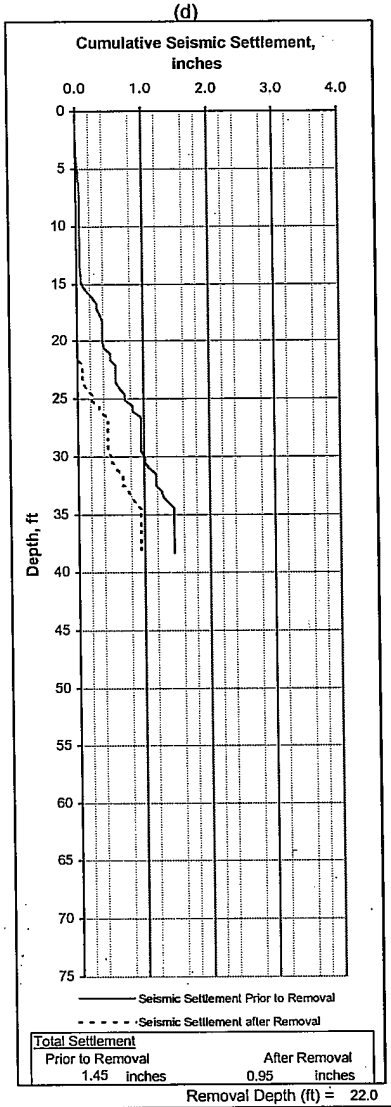

Removal Depth (ft) = 22.0

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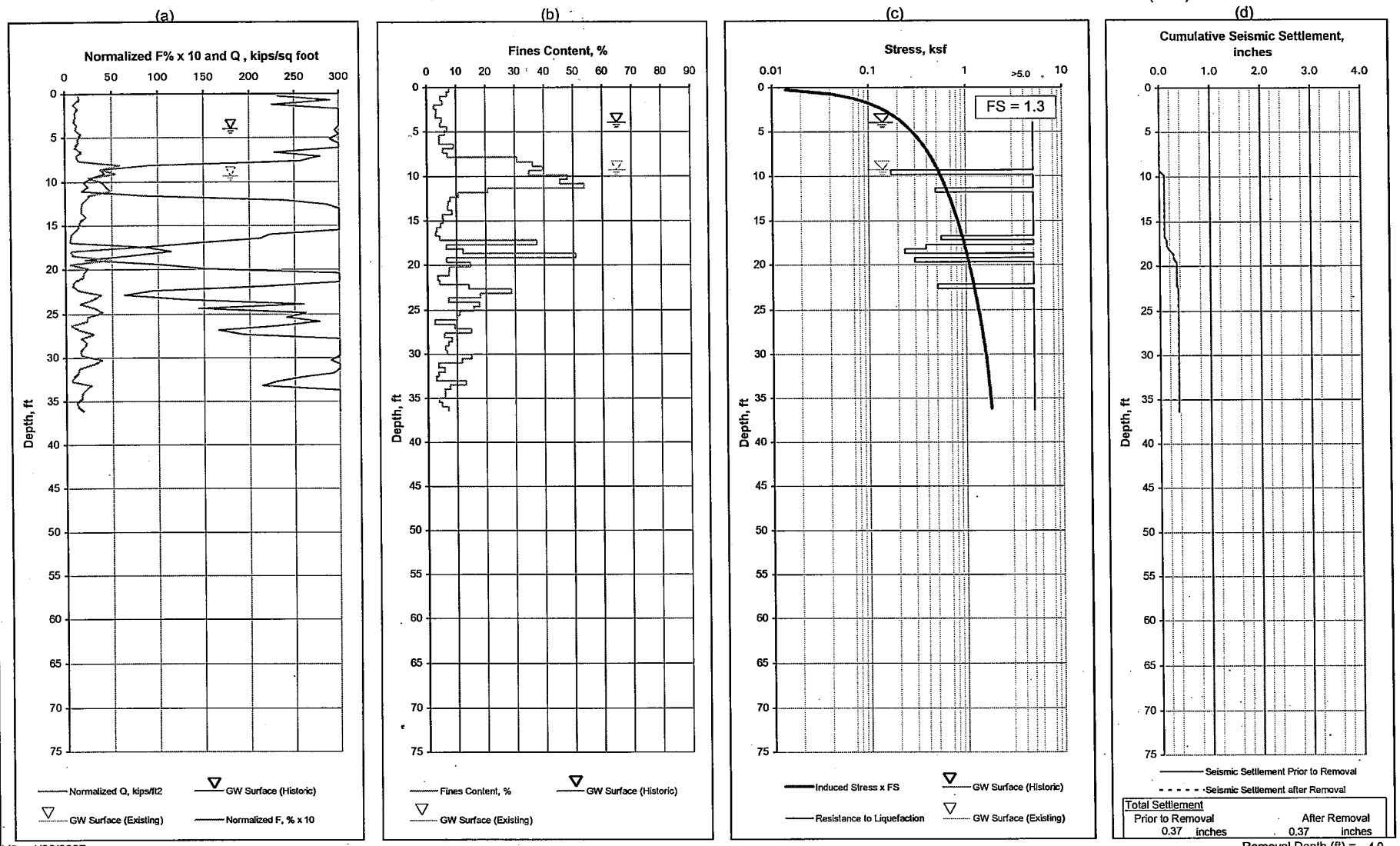
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5). 0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)... 0.83 g	Figure No.	C7
For: Newhall Land & Farming Co.	Magnification Factor..... 1	Date.	12-21-2009

Location..... **CPT-101**
Elevation (MSL)..... 914 ft



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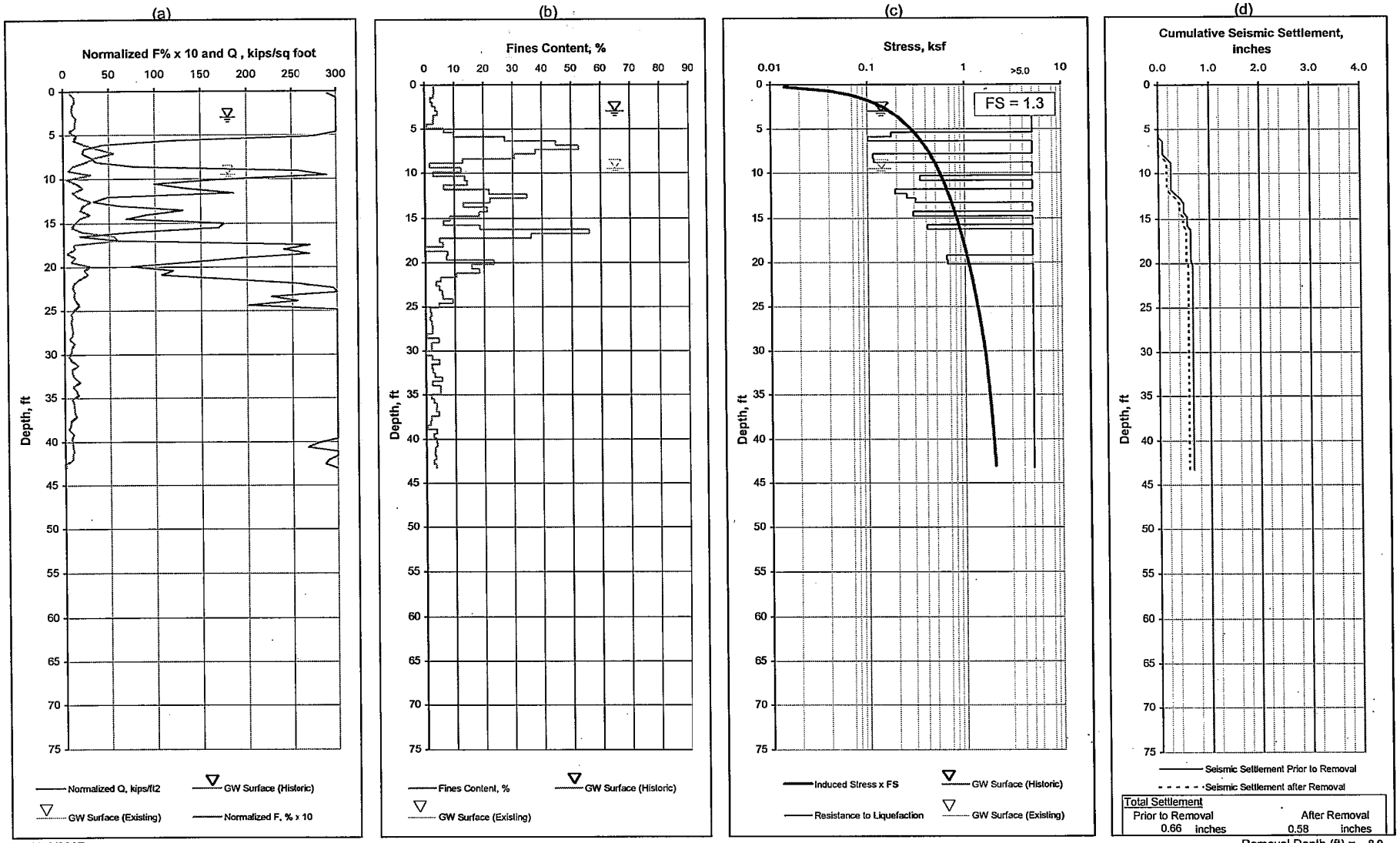
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C8
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-103
Elevation (MSL)..... 905 ft



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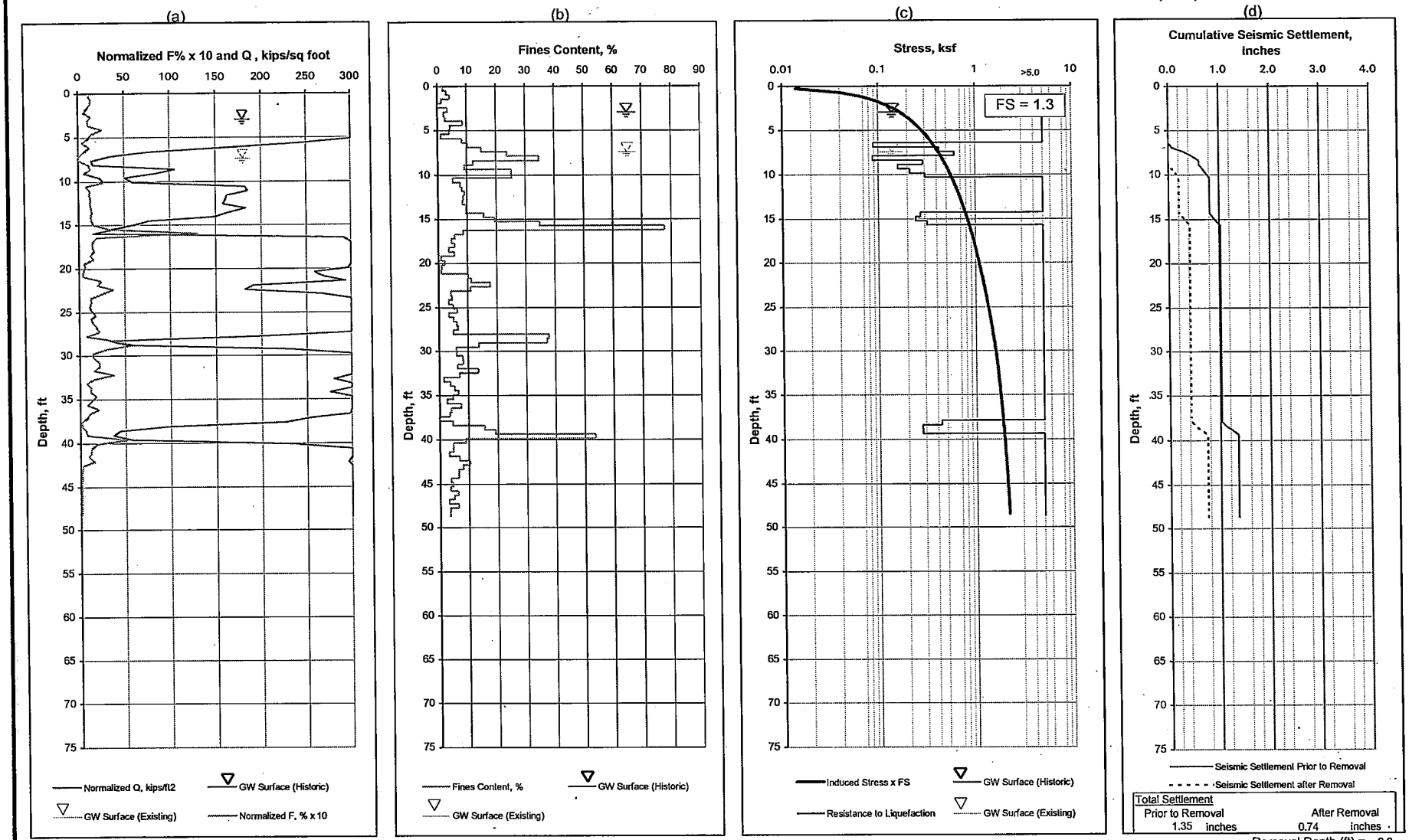
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C9
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-105
Elevation (MSL)..... 907 ft



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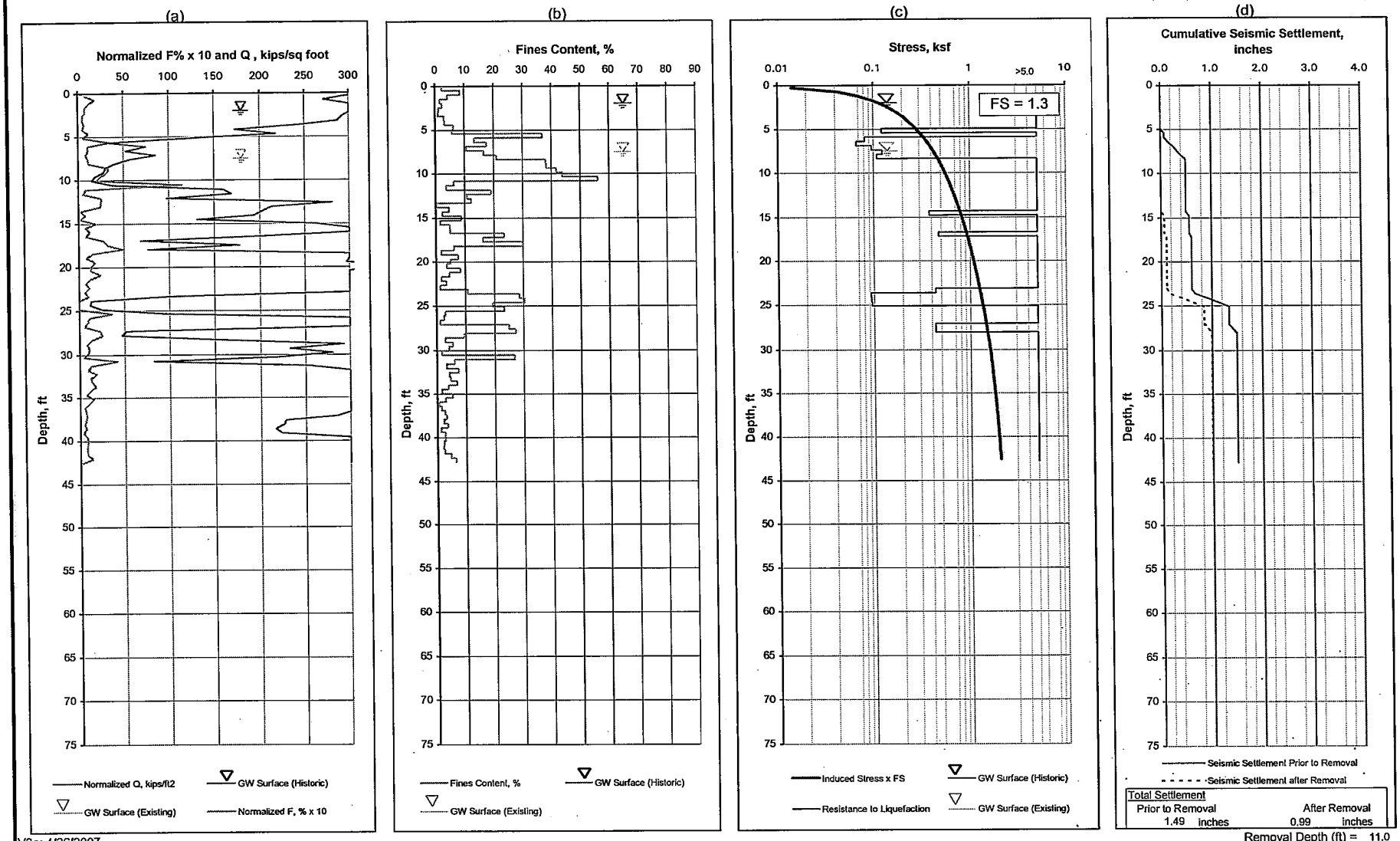
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C10
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-106
Elevation (MSL)..... 905 ft



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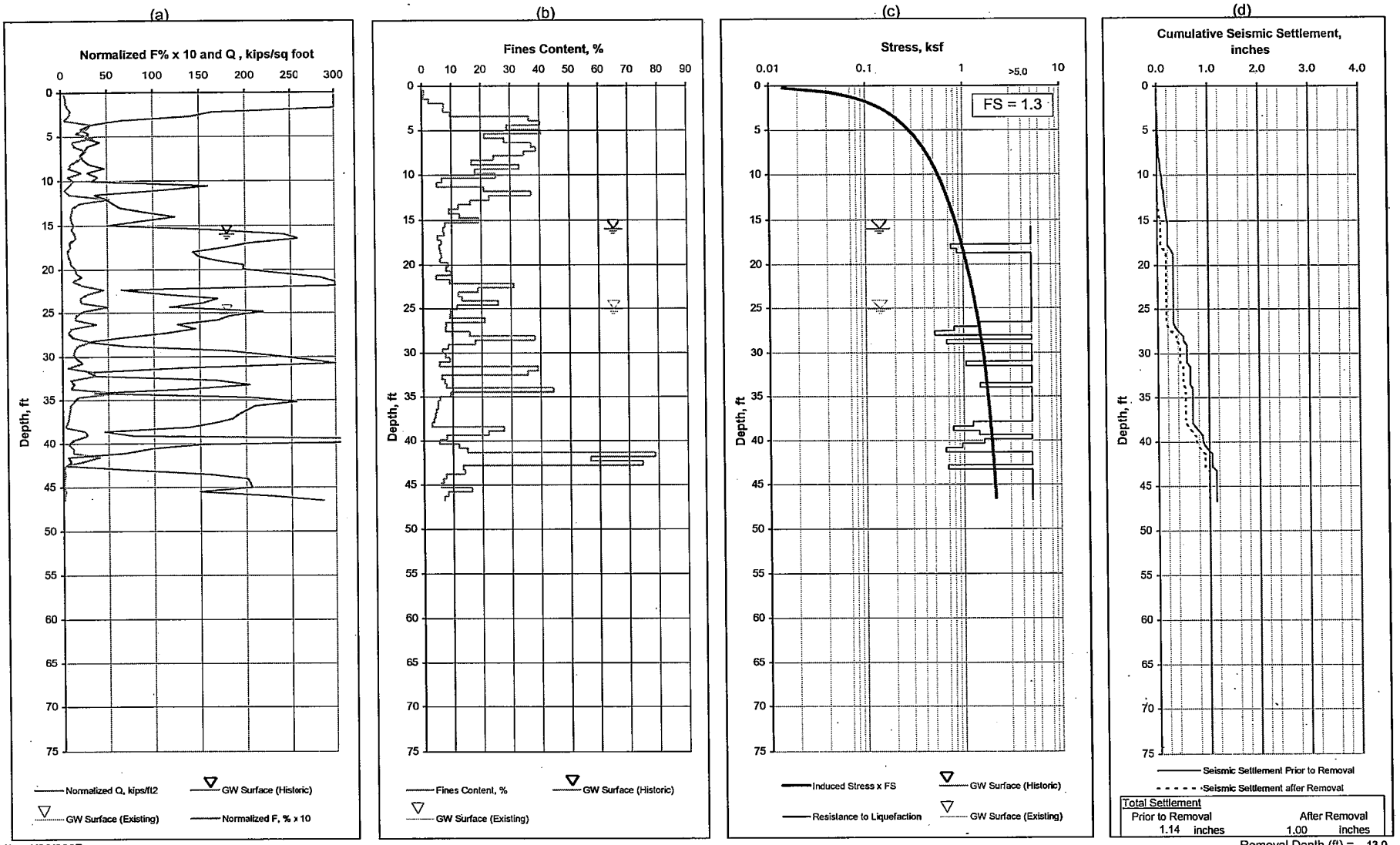
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C11
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-108
Elevation (MSL)..... 951 ft



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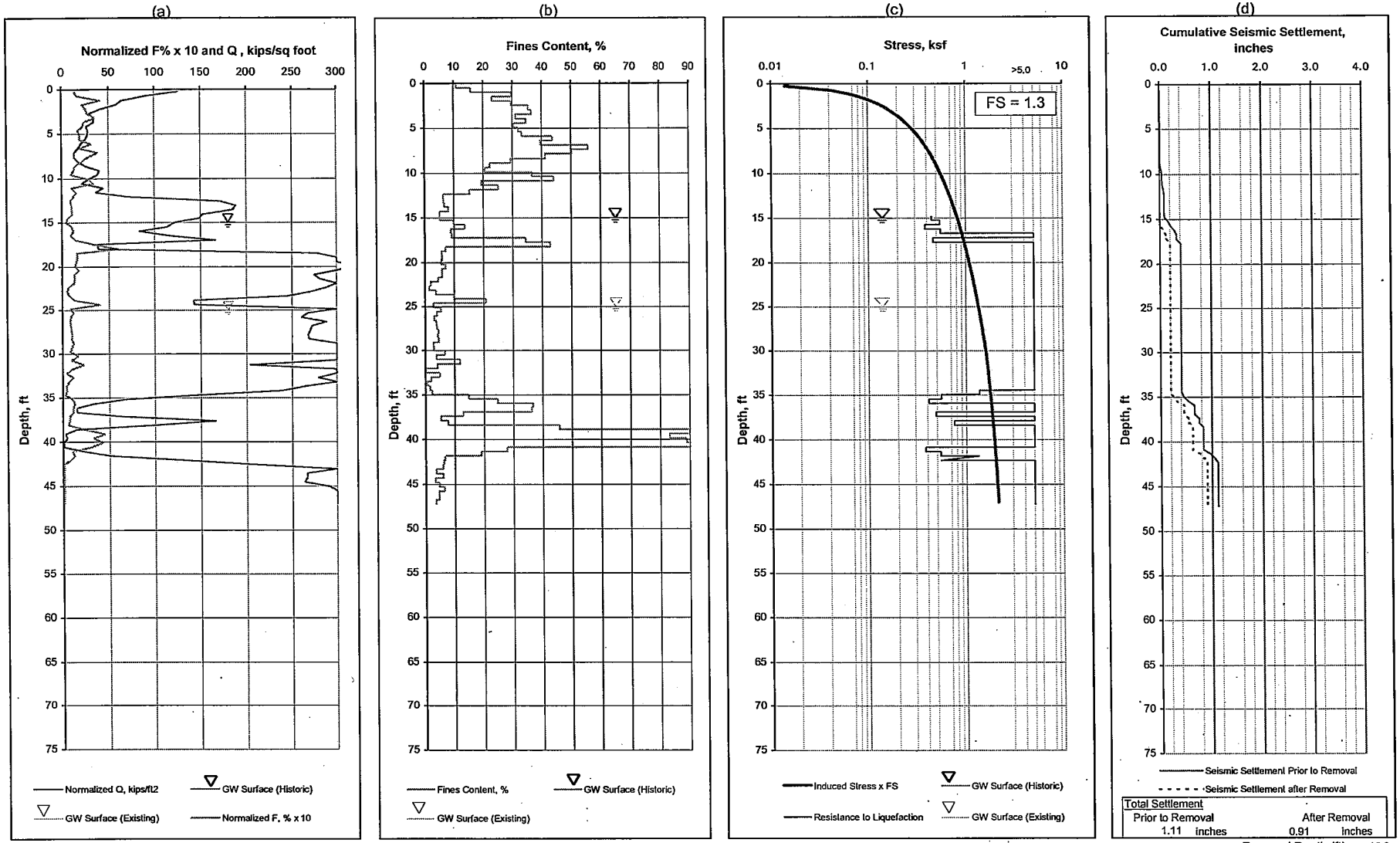
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

Riverwood Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C12
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-109**
Elevation (MSL)..... 952 ft



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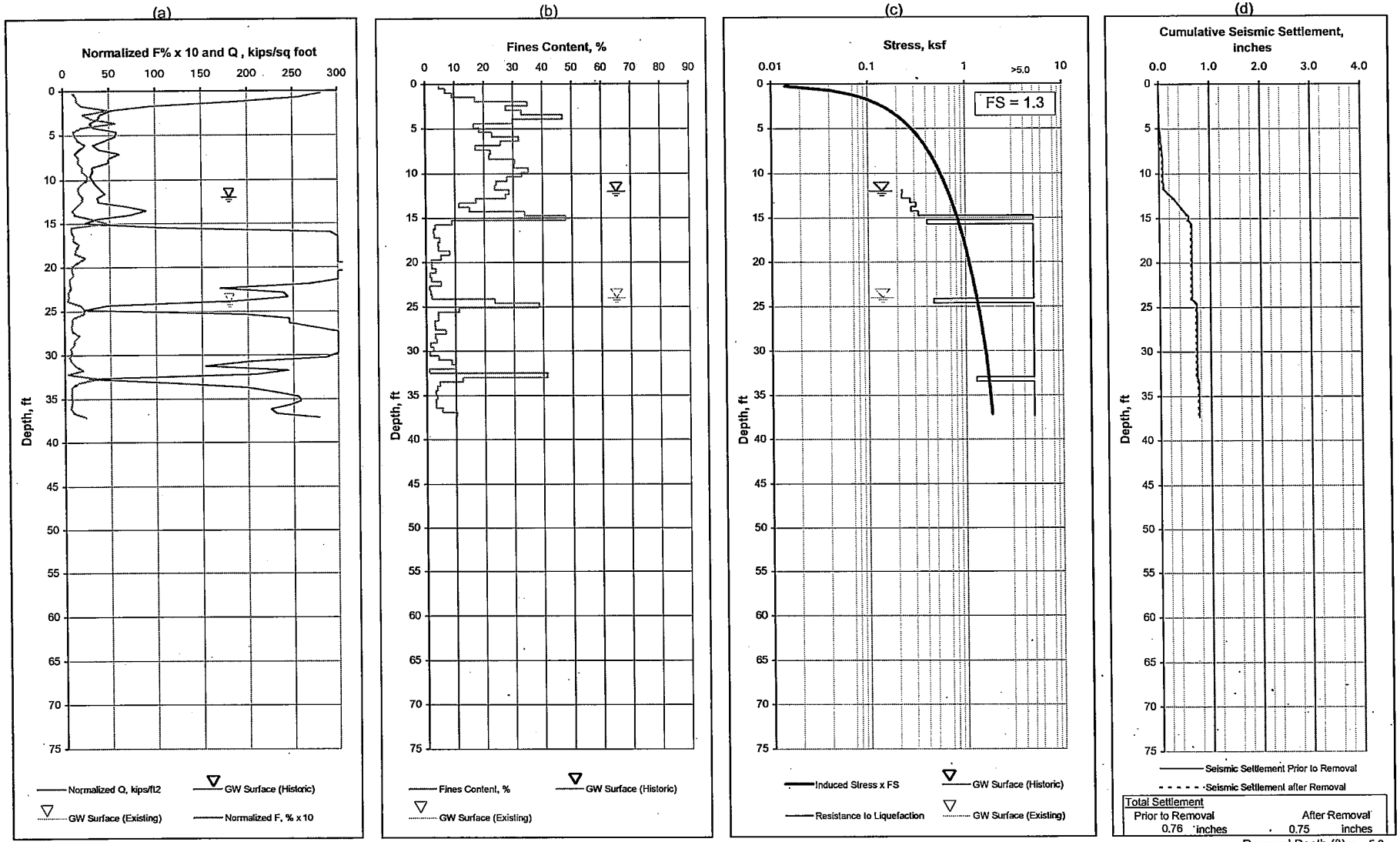
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C13
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... CPT-110
Elevation (MSL)..... 953 ft



V3e; 4/26/2007

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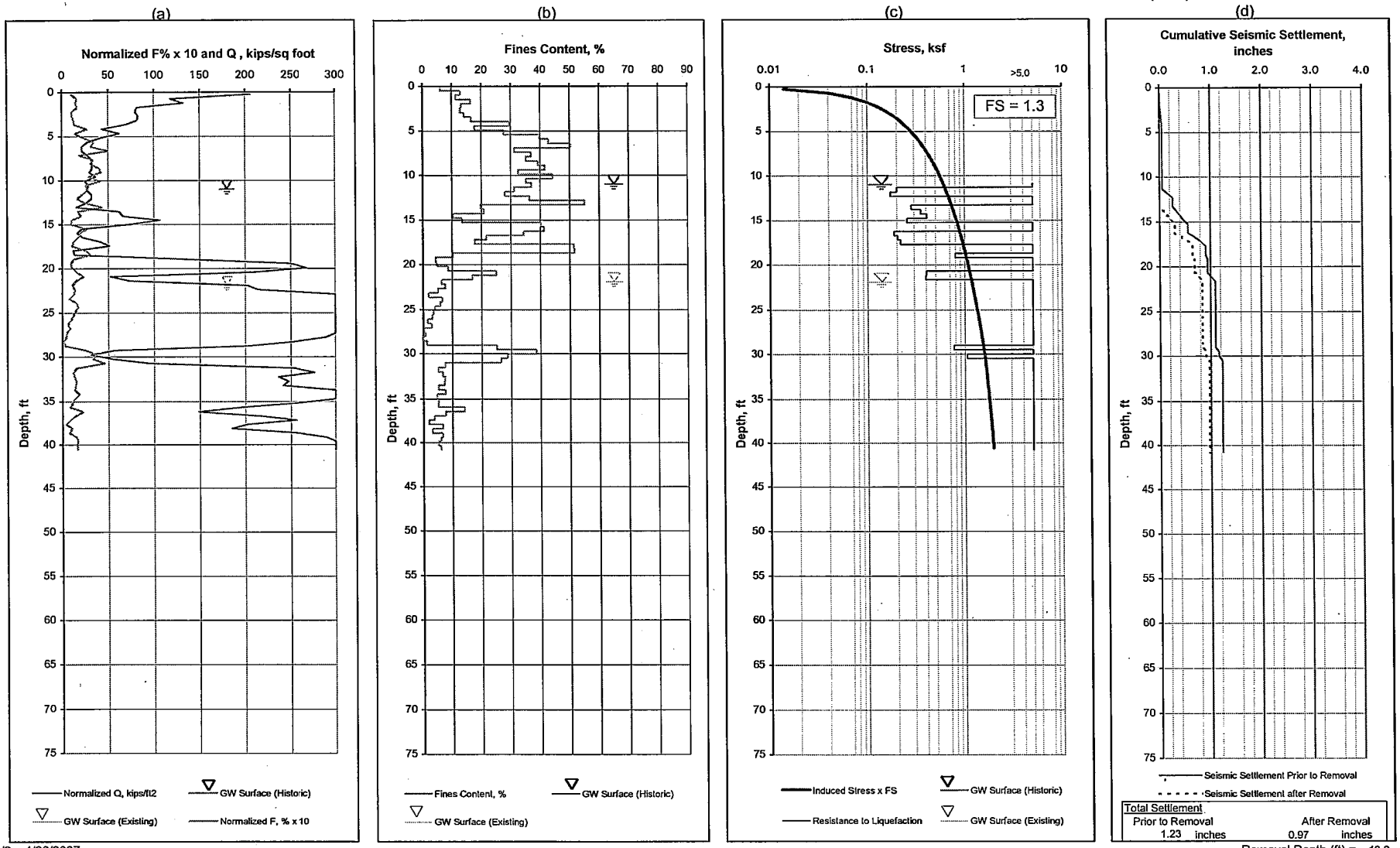
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C14
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-111**
Elevation (MSL)..... 953 ft



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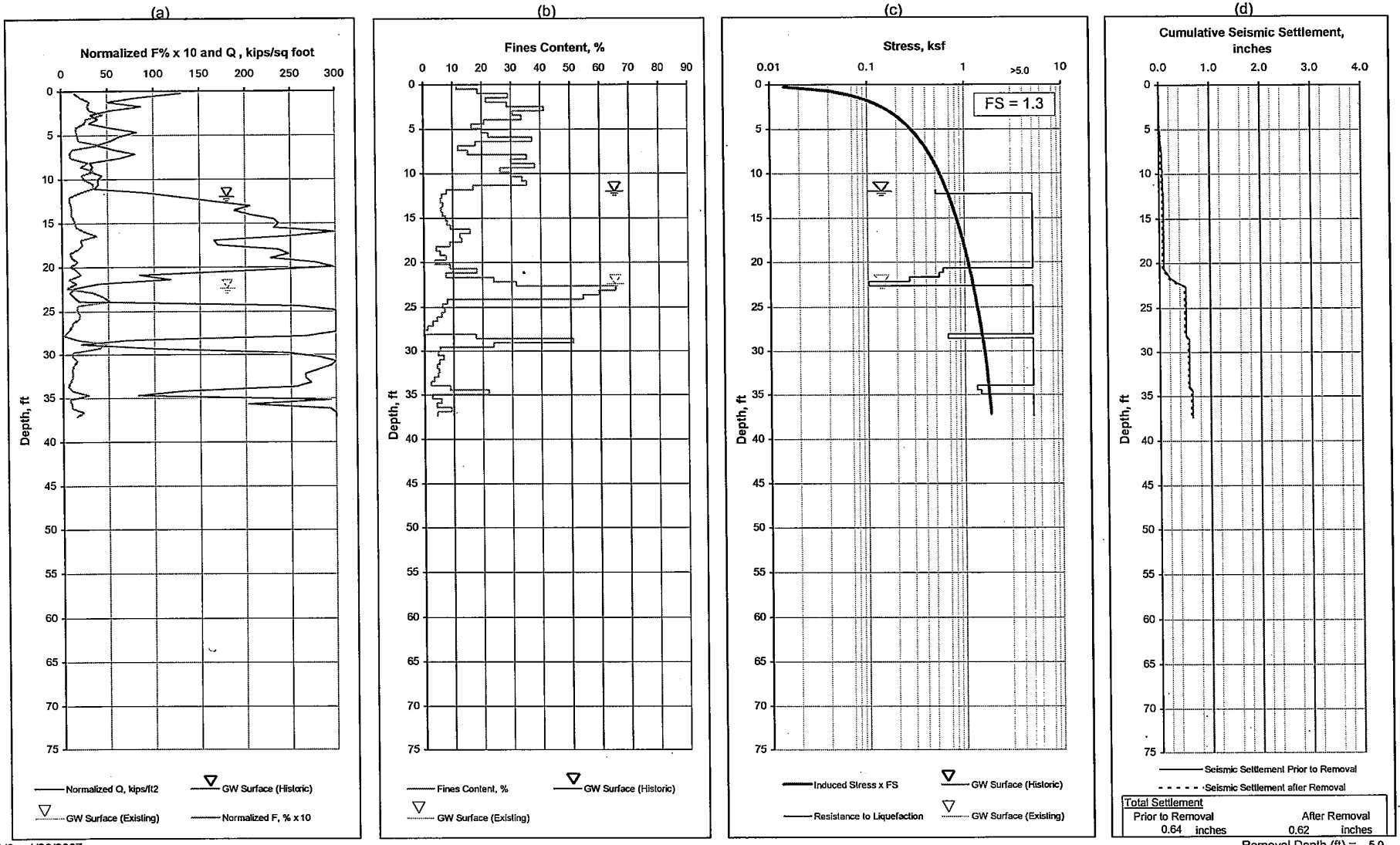
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108		Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California		Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C15
For: Newhall Land & Farming Co.		Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-114**
Elevation (MSL)..... 954 ft



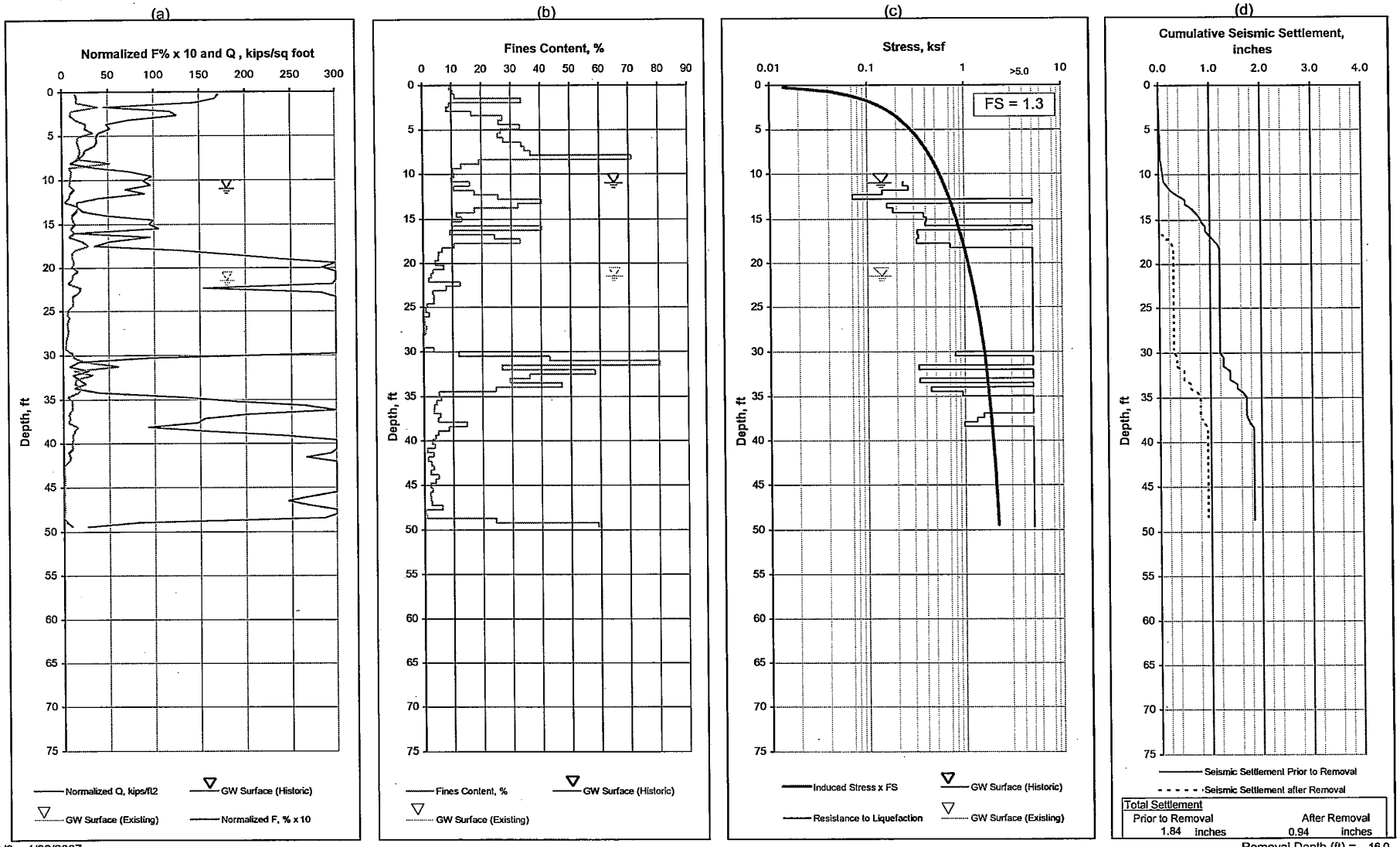
V3e; 4/26/2007

Allan E. Seward Engineering Geology, Inc.
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C16
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-115**
Elevation (MSL)..... 954 ft



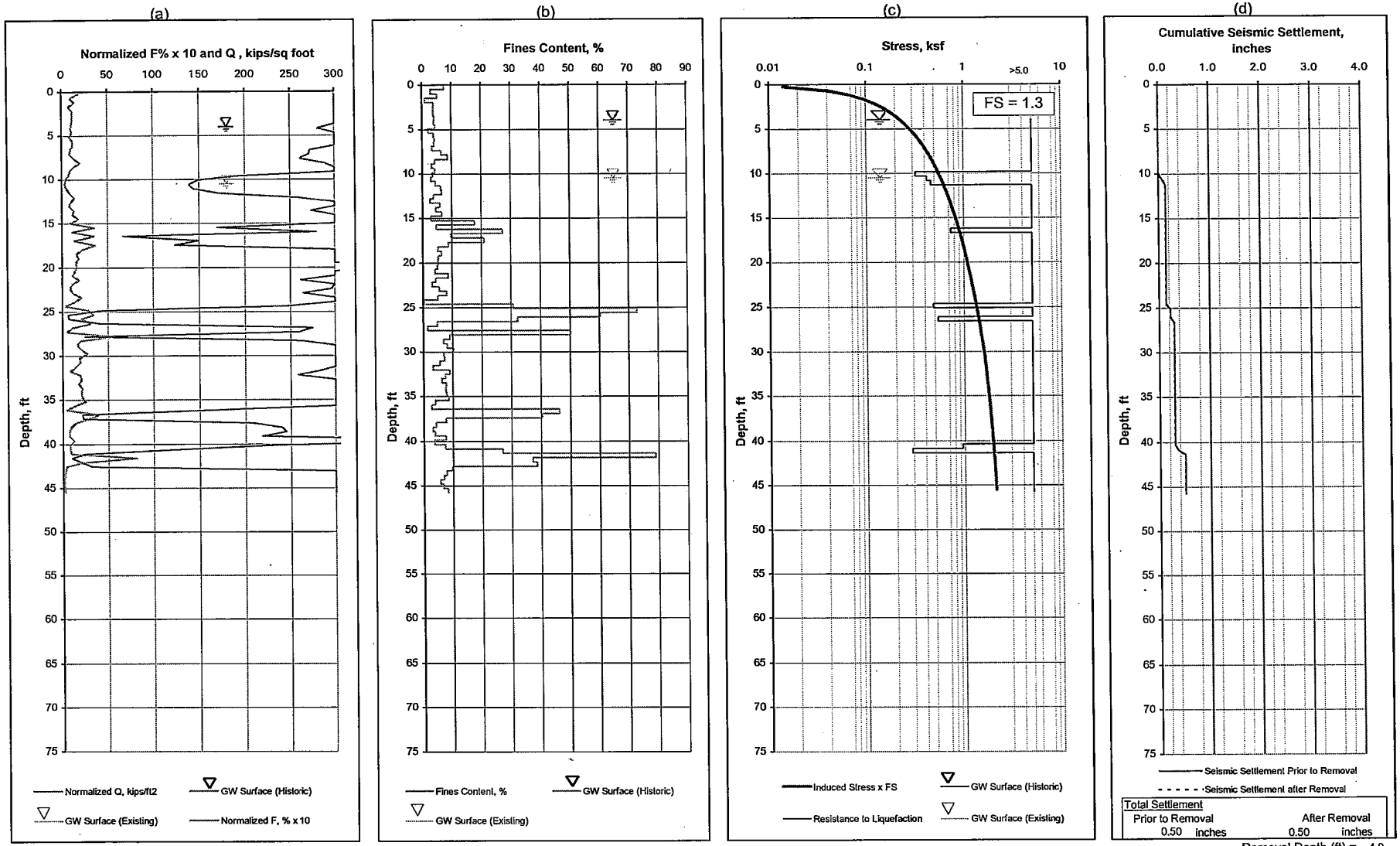
V3e; 4/26/2007

Allan E. Seward Engineering Geology, Inc.
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5). 0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)... 0.83 g	Figure No.	C17
For: Newhall Land & Farming Co.	Magnification Factor..... 1	Date.	12-21-2009

Location..... **CPT-118**
Elevation (MSL)..... 920 ft



V3e; 4/26/2007

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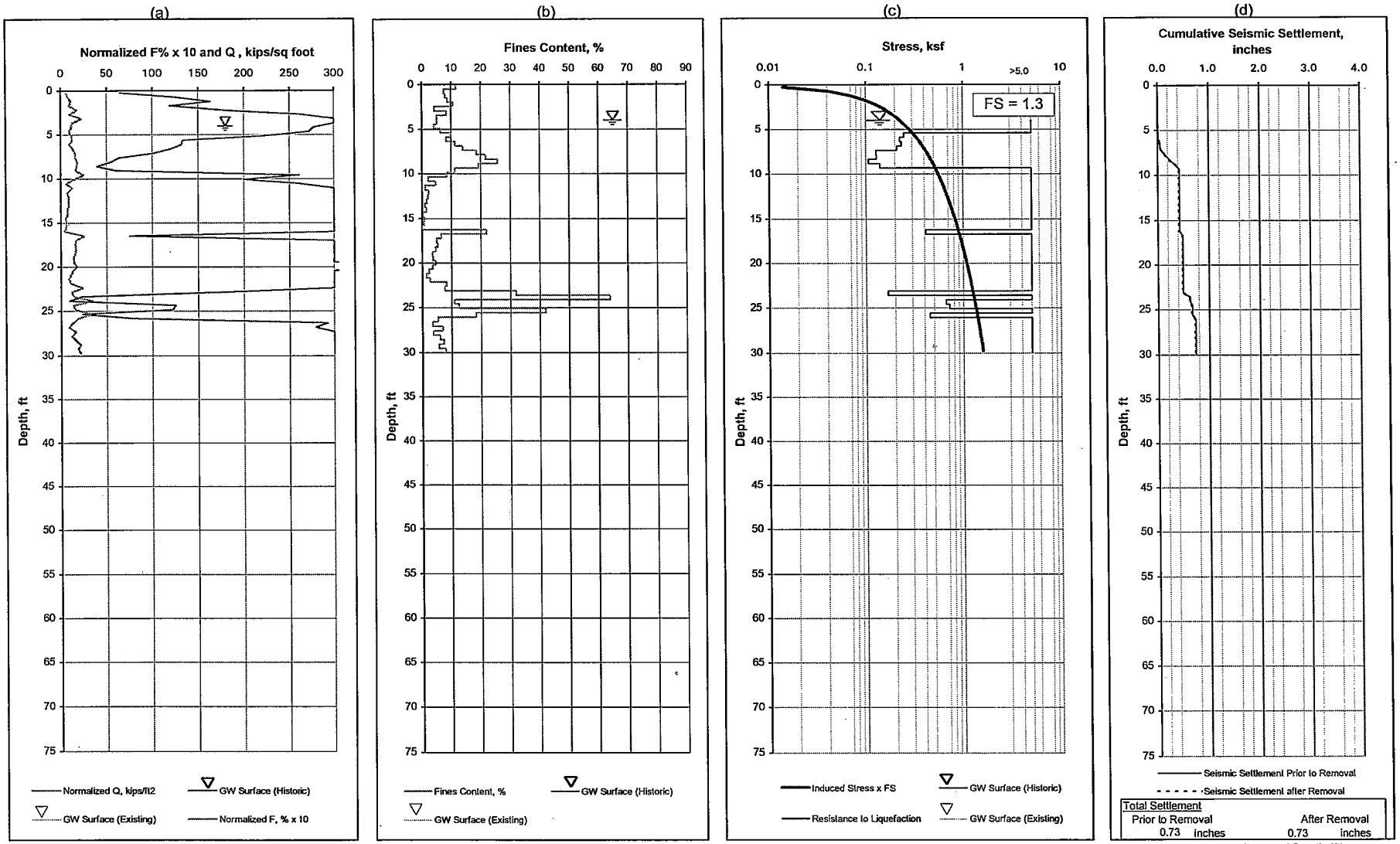
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108		Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California		Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C18
For: Newhall Land & Farming Co.		Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-122A**
Elevation (MSL)..... 906 ft



V3e; 4/26/2007

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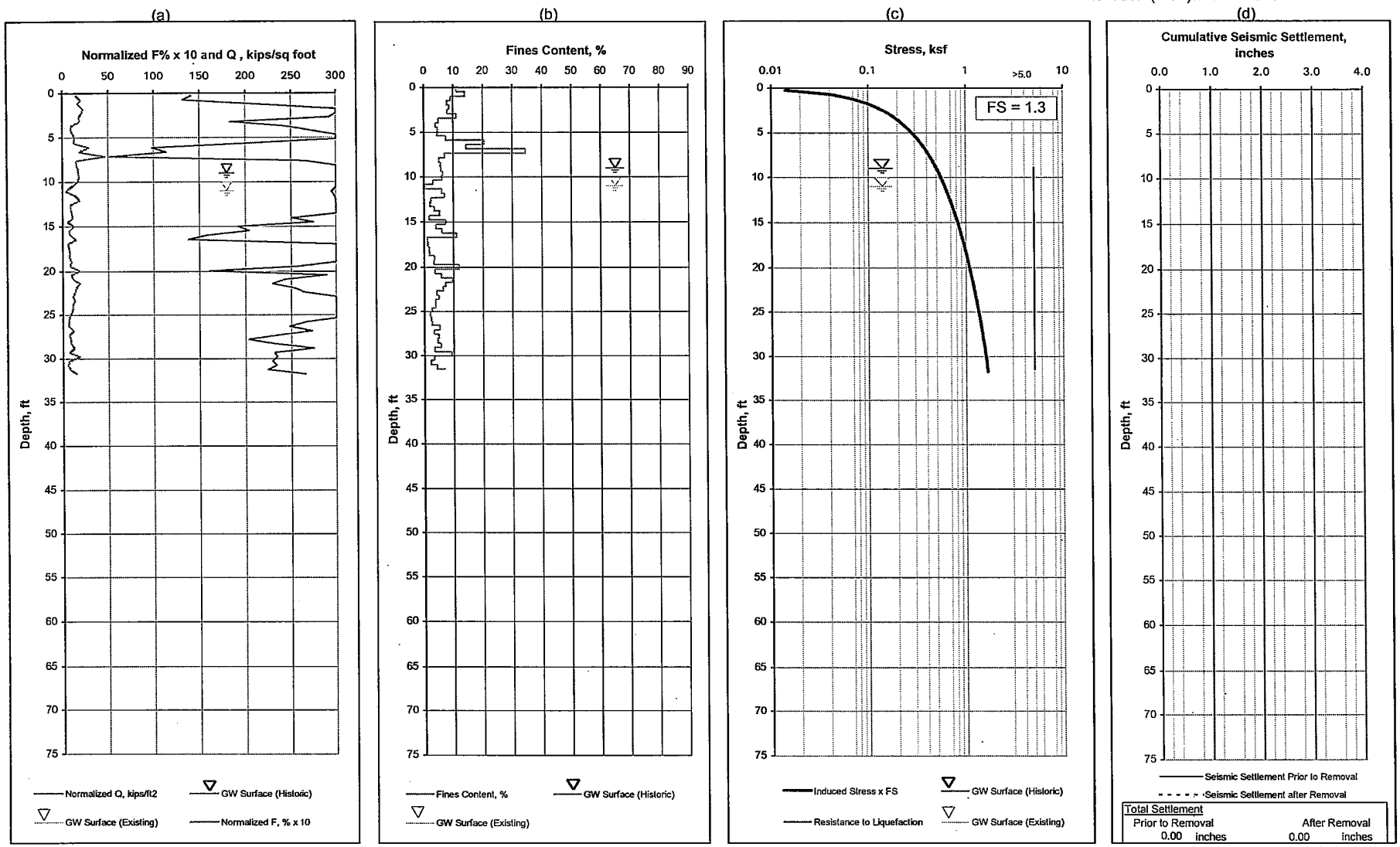
Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5). 0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)... 0.83 g	Figure No.	C19
For: Newhall Land & Farming Co.	Magnification Factor..... 1	Date.	12-21-2009

Location..... CPT-125
Elevation (MSL)..... 913 ft



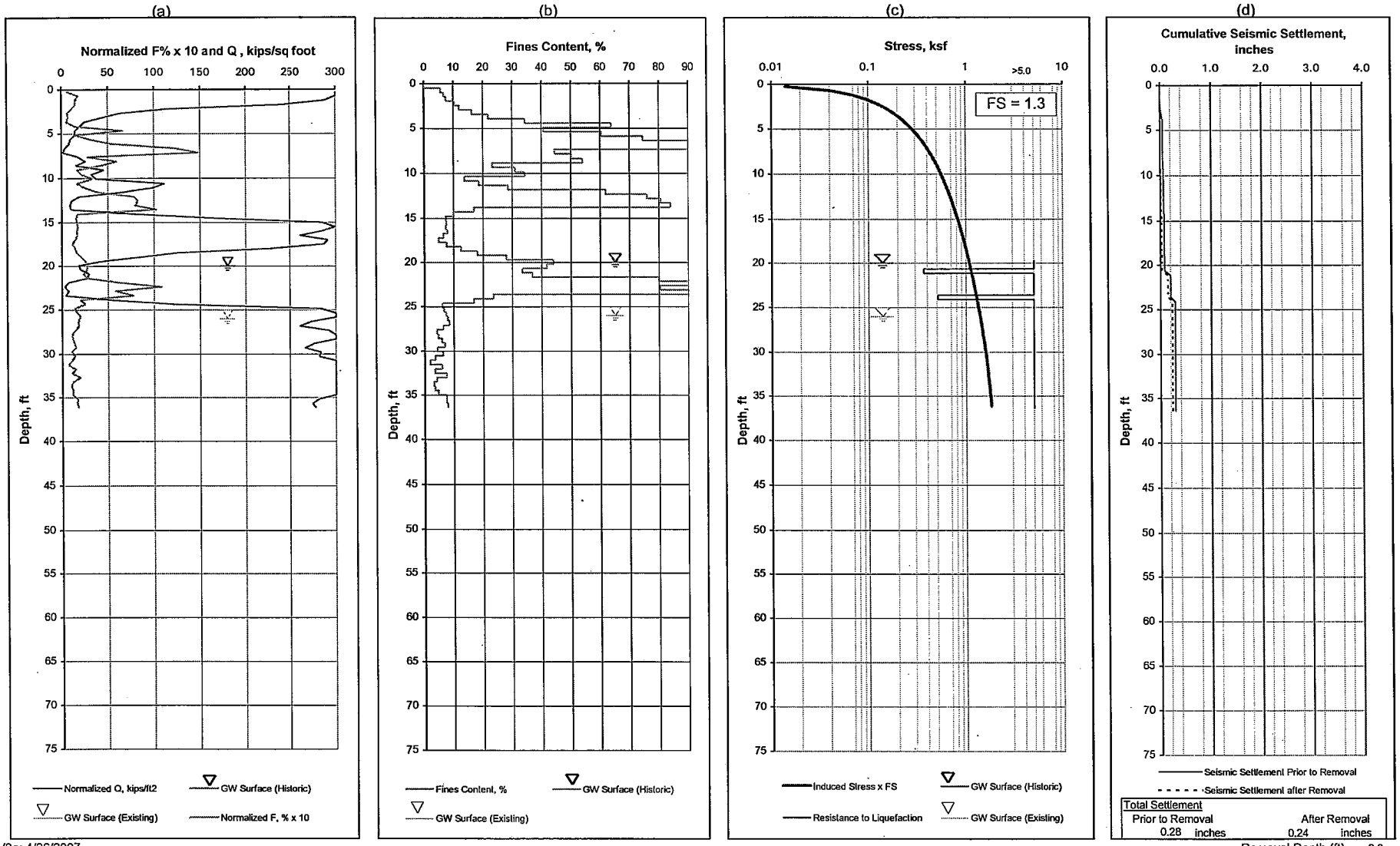
V3e; 4/26/2007

Allan E. Seward Engineering Geology, Inc.
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5).	0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)...	0.83 g	Figure No.	C20
For: Newhall Land & Farming Co.	Magnification Factor.....	1	Date.	12-21-2009

Location..... **CPT-128**
Elevation (MSL)..... 951 ft



V3e; 4/26/2007

Allan E. Seward Engineering Geology, Inc.
Geological and Geotechnical Consultants

Analysis of Liquefaction Potential Based on q_c and f_s from CPT Data

River Village, VTT 53108	Weighted Gr. Acc. (M=7.5). 0.56 g	Project No.	1702R-4
Castaic, L A County, California	Design Gr. Acc.(M= 6.5)... 0.83 g	Figure No.	C21
For: Newhall Land & Farming Co.	Magnification Factor..... 1	Date.	12-21-2009

LIQUEFACTION POTENTIAL AND CYCLIC SETTLEMENT ANALYSIS

PROJECT: River Village, VTT 63108
Job No.: 1702R-4

Location	CPT-115	Basic Earthquake Magnitude	7.5	Conditions:	Existing conditions with historic groundwater level
Type of Sampler (CPT/SPT/CD)	CPT	Approx. Distance From Site (miles)	2.5		
Ground Surface Elevation	954 ft. MSL	Design Magnitude	6.5	1.30	Specified Factor of Safety
Existing Ground Water Depth	21.5 ft. Below GL	Peak Ground Accel (M =7.5)	0.50 g.		FS = 1.3
Historic Ground Water Depth	11.0 ft. Below GL	Design PGA (M = 6.5)	0.87 g.		
		Magnification Factor	1.0		

Layer No.	Mid-layer Depth (ft)	Elevation (ft MSL)	Depth of layer below Ground Level (ft) Top	Bottom	Total Unit Wt. (1) (kcf)	Total Stress (2) (ksf)	Effective Stress (3) (ksf)	CPT Tip Resistance (4) (ksf)	F, % normalized (5)	1st trial I_c (6)	2nd trial I_c (7)	exponent n of Final I_c and q_{c1N} (8)	q_{c1N} normalized (9) (ksf)
1	0.25	953.8	0	0.49	0.120	0.029	0.029	42.66	1.451	1.42	1.65	0.50	171.00
2	0.74	953.3	0.40	0.98	0.120	0.088	0.088	72.24	1.832	1.54	1.90	0.50	167.18
3	1.23	952.8	[illegible]	[illegible]	0.120	0.148	0.148	81.12	[illegible]	1.50	1.97	0.50	145.12
4	1.72	952.3	1.48	1.97	0.120	0.207	0.207	30.30	[illegible]	2.26	2.56	0.50	45.77
5	2.22	951.8	1.97	2.46	0.120	0.266	0.266	65.20	0.861	1.54	1.65	0.50	117.58
6	2.71	951.3	2.46	2.95	0.120	0.325	0.325	101.72	0.692	1.52	1.80	0.50	125.12
7	3.20	950.8	2.95	3.44	0.120	0.383	0.383	65.50	1.479	1.86	2.13	0.50	72.48
8	3.69	950.3	3.44	3.94	0.120	0.443	0.443	47.00	2.574	2.16	2.42	0.50	48.54
9	4.18	949.8	3.94	4.43	0.120	0.502	0.502	53.92	2.513	2.17	2.38	0.50	57.20
10	4.68	949.3	4.43	4.92	0.120	0.561	0.561	33.98	3.336	2.44	2.64	0.70	40.16
11	5.17	948.8	4.92	5.41	0.120	0.620	0.620	47.44	1.756	2.20	2.40	0.50	[illegible]
12	5.66	948.3	5.41	5.90	0.120	0.679	0.679	45.30	1.655	2.19	2.36	0.50	[illegible]
13	6.16	947.8	5.91	6.40	0.120	0.739	0.739	44.07	1.600	2.20	2.42	0.50	[illegible]
14	6.65	947.4	6.40	6.89	0.120	0.797	0.797	26.66	1.926	2.47	2.63	0.70	[illegible]
15	7.14	946.9	6.89	7.38	0.120	0.856	0.856	20.00	1.013	2.50	2.65	0.70	23.14
16	7.63	946.4	7.38	7.87	0.120	0.915	0.915	17.46	1.572	2.63	2.83	1.00	18.30
17	8.12	945.9	7.87	8.37	0.120	0.974	0.974	9.78	5.220	3.17	3.17	1.00	9.04
18	8.62	945.4	8.37	8.86	0.120	1.034	1.034	53.06	0.785	2.00	2.21	0.50	56.27
19	9.11	944.9	8.86	9.35	0.120	1.093	1.020	144.30	0.660	1.00	2.00	0.50	75.15
20	9.60	944.4	9.35	9.84	0.120	1.151	1.151	151.60	0.617	1.60	1.90	0.50	97.15
21	10.09	943.9	9.84	10.33	0.120	1.210	1.210	141.16	0.647	1.62	1.91	0.50	88.19
22	10.58	943.4	10.33	10.83	0.120	1.270	1.270	157.60	0.617	1.70	1.87	0.50	96.13
23	11.07	942.9	10.83	11.32	0.120	1.329	1.324	145.58	1.295	2.03	2.11	0.50	69.03
24	11.57	942.4	11.32	11.81	0.120	1.388	1.353	151.42	0.883	1.64	1.91	0.50	89.54
25	12.06	941.9	11.81	12.30	0.120	1.447	1.381	68.66	0.715	2.08	2.15	0.50	40.16
26	12.55	941.5	12.30	12.80	0.120	1.506	1.409	27.39	0.307	2.32	2.38	0.50	15.82
27	13.05	941.0	12.80	13.29	0.120	1.565	1.438	24.15	1.741	2.70	2.70	1.00	15.00
28	13.54	940.5	13.29	13.78	0.120	1.624	1.466	38.60	1.512	2.48	2.54	0.50	33.15
29	14.03	940.0	13.78	14.27	0.120	1.683	1.494	89.70	0.600	2.10	2.10	0.50	49.31
30	14.52	939.5	14.27	14.76	0.120	1.742	1.522	174.08	1.101	1.09	1.94	0.50	99.15
31	15.01	939.0	14.76	15.26	0.120	1.801	1.551	171.70	1.344	1.97	2.01	0.50	94.78
32	15.51	938.5	15.26	15.75	0.120	1.861	1.579	191.80	0.621	1.83	1.87	0.50	114.84
33	16.00	938.0	15.75	16.24	0.120	1.919	1.608	23.12	1.270	2.70	2.70	1.00	[illegible]
34	16.49	937.5	16.24	16.73	0.120	1.978	1.636	177.18	0.738	1.81	1.85	0.50	[illegible]
35	16.98	937.0	16.73	17.22	0.120	2.037	1.664	97.34	2.105	2.31	2.35	0.50	[illegible]
36	17.47	936.5	17.22	17.72	0.120	2.096	1.693	65.68	2.100	2.52	2.55	0.50	34.70
37	17.97	936.0	17.72	18.21	0.120	2.156	1.721	244.24	1.204	1.08	1.91	0.50	127.95
38	18.46	935.5	18.21	18.70	0.120	2.215	1.749	350.72	0.996	1.69	1.71	0.50	[illegible]
39	18.95	935.1	18.7	19.19	0.120	2.275	1.778	450.97	1.045	1.65	1.65	0.50	237.62
40	19.44	934.6	19.19	19.69	0.120	2.333	1.806	593.74	1.245	[illegible]	1.64	0.50	303.38
41	19.94	934.1	19.69	20.18	0.120	2.392	1.835	582.87	0.994	[illegible]	1.50	0.50	288.58
42	20.43	933.6	20.18	20.67	0.120	2.451	1.863	603.90	1.047	1.73	1.74	0.50	[illegible]
43	20.92	933.1	20.67	21.16	0.120	2.510	1.891	801.49	1.103	1.52	1.53	0.50	[illegible]
44	21.41	932.6	21.16	21.65	0.120	2.569	[illegible]	821.52	0.963	1.49	1.49	0.50	407.50
45	21.90	932.1	21.65	22.15	0.120	2.628	[illegible]	[illegible]	0.642	1.47	1.43	0.50	[illegible]
46	22.40	931.6	22.15	22.64	0.120	2.687	[illegible]	302.62	1.054	2.07	1.96	0.50	153.72
47	22.89	931.1	22.64	[illegible]	0.120	2.746	[illegible]	673.24	1.787	[illegible]	1.70	0.50	263.72

Layer No.	Final I_c (10)	K_c factor for fines (11)	Equivalent $(q_{c1N})_{cs}$ clean sand (12)	Percent Fines, % Estimated (13)	Lab (14)	Cyclic Resistance Ratio (15) M=7.5	Cyclic Resistance M=7.5 (16) (ksf)	Stress Reduction Factor, r_d (17)	Specified FS x Induced Cyclic Stress (18) M=7.5 (ksf)	Calculated Factor of Safety (19)	Cumulative Settlement after Removals (inches)	Thickness of Potentially Liquefiable Layers After Removals (ft)
1	1.65	1.15	196.44	9		>0.6	>5.0	0.998	[illegible]	Non Liquefiable	0.00	
2	1.90	1.19	198.64	10		>0.6	>5.0	0.996	0.044	Non Liquefiable	0.00	
3	1.97	1.21	175.97	11		>0.6	>5.0	0.997	0.073	Non Liquefiable	0.00	
4	2.56	3.10	141.82	34		[illegible]	0.071	0.996	[illegible]	[illegible]	0.00	
5	1.65	1.15	134.81	9		[illegible]	[illegible]	0.995	[illegible]	0.61	0.00	
6	1.80	1.11	138.64	[illegible]		[illegible]	[illegible]	0.994	0.161	0.66	0.00	
7	2.13	1.51	[illegible]	17		0.201	[illegible]	0.993	0.190	0.53	0.00	
8	2.42	[illegible]	115.89	27		0.224	[illegible]	0.991	0.219	0.59	0.00	
9	2.38	2.25	117.00	26		0.233	0.117	0.990	0.248	0.61	0.00	
10	2.64	3.05	122.60	30		0.252	0.141	0.989	0.277	0.66	0.00	
11	2.40	2.32	115.85	28		0.139	[illegible]	0.988	0.305	0.37	0.00	
12	2.36	2.22	114.05	25		0.135	[illegible]	0.987	0.334	0.36	0.00	
13	2.42	2.41	114.96	27		0.137	[illegible]	0.986	0.363	0.35	0.00	
14	2.63	[illegible]	[illegible]	34		0.123	[illegible]	0.985	[illegible]	0.33	0.00	
15	2.65	3.24	[illegible]	35		0.119	[illegible]	0.985	0.420	0.32	0.00	
16	2.83	3.50	Fine Grained	37		>0.6	>5.0	0.982	0.448	Non Liquefiable	0.00	
17	3.17	[illegible]	Fine Grained	71		>0.6	>5.0	0.981	0.477	Non Liquefiable	0.00	
18	2.21	1.65	[illegible]	10		[illegible]	[illegible]	0.980	[illegible]	0.27	0.00	
19	2.00	1.31	98.15	13		[illegible]	[illegible]	0.979	[illegible]	0.45	0.00	
20	1.90	1.10	[illegible]	10		[illegible]	[illegible]	0.979	0.561	0.59	0.00	
21	1.91	1.20	[illegible]	11		[illegible]	[illegible]	0.976	0.589	0.51	0.00	
22	1.87	1.16	[illegible]	10		[illegible]	[illegible]	0.975	0.617	0.56	0.00	
23	2.11	1.47	[illegible]	[illegible]		[illegible]	[illegible]	0.974	0.645	0.47	0.00	0.49
24	1.91	1.20	107.27	11		0.195	[illegible]	0.073	[illegible]	0.51	0.00	0.49
25	2.15	[illegible]	[illegible]	[illegible]		[illegible]	0.142	0.072	0.701	0.26	0.00	0.49
26	2.38	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	0.071	0.729	0.13	0.00	0.5
27	2.70	3.97	Fine-Grained	40		>0.6	>5.0	0.070	0.757	Non Liquefiable	0.00	-
28	2.54	2.96	[illegible]	[illegible]		[illegible]	[illegible]	0.968	0.784	[illegible]	0.00	0.49
29	2.15	1.55	[illegible]	10		[illegible]	[illegible]	0.967	[illegible]	0.29	0.00	0.49
30	1.94	1.23	[illegible]	11		0.246	[illegible]	0.966	[illegible]	0.50	0.00	0.49
31	2.01	1.32	[illegible]	13		0.200	[illegible]	0.965	[illegible]	0.61	0.00	0.5
32	1.87	1.17	[illegible]	10		0.250	[illegible]	0.964	0.894	0.57	0.00	0.40
33	2.70	[illegible]	Fine Grained	40		>0.6	>5.0	0.963	[illegible]	Non Liquefiable	0.00	-
34	1.85	1.14	108.72	9		[illegible]	[illegible]	0.962	0.948	0.45	0.07	0.49
35	2.35	[illegible]	[illegible]	24		0.201	0.334	0.960	0.975	0.44	0.16	0.49
36	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	1.003	0.41	0.24	0.5
37	1.91	1.10	152.73	11		0.411	0.700	[illegible]	1.030	0.80	0.28	0.49
38	1.71	[illegible]	[illegible]	[illegible]		>0.6	>5.0	0.957	1.057	Non Liquefiable	[illegible]	-
39	1.65	1.31	[illegible]	5		>0.6	>5.0	0.956	[illegible]	Non Liquefiable	[illegible]	-
40	1.64	1.00	[illegible]	5		>0.6	>5.0	0.955	[illegible]	Non Liquefiable	0.28	-
41	1.50	1.00	[illegible]	4		>0.6	>5.0	0.954	1.137	Non Liquefiable	0.28	-
42	1.74	1.07	[illegible]	7		>0.6	>5.0	[illegible]	1.164	Non Liquefiable	0.28	-
43	1.53	1.00	[illegible]	3		>0.6	>5.0	[illegible]	[illegible]	Non Liquefiable	0.28	-
44	1.49	1.00	[illegible]	[illegible]		>0.6	>5.0	[illegible]	1.217	Non Liquefiable	0.28	-
45	1.43	1.00	[illegible]	2		>0.6	>5.0	0.949	1.243	Non Liquefiable	0.28	-
46	1.96	1.28	[illegible]	12		>0.5	>5.0	0.948	1.270	Non Liquefiable	0.28	-
47	1.70	[illegible]	[illegible]	6		>0.5	>5.0	0.947	1.296	Non Liquefiable	0.28	-

LIQUEFACTION POTENTIAL AND CYCLIC SETTLEMENT ANALYSIS

Layer No.	Mid-layer Depth (ft)	Elevation (ft. MSL)	Depth of layer below Ground Level (ft) Top	Bottom	Total Unit Wt. (1) (kcf)	Total Stress (2) (ksf)	Effective Stress (3) (ksf)	CPT Tip Resistance (4) (ksf)	F, % normalized (5)	1st trial I_c (6)	2nd trial I_c (7)	exponent n of Final I_c and q_{c1N} (8)	q_{c1N} normalized (9) (ksf)	Final I_c (10)	K_c factor for fines (11)	Equivalent $(q_{c1N})_{cs}$ clean sand (12)	Percent Fines, % Estimated (13)	Lab (14)	Cyclic Resistance Ratio (15) M=7.5	Cyclic Resistance M = 7.5 (16) (ksf)	Stress Reduction Factor, r_d (17)	Specified FS x Induced Cyclic Stress (18) M=7.5 (ksf)	Calculated Factor of Safety (19)	Cumulative Settlement after Removals (inches)	Thickness of Potentially Liquefiable Layers After Removals (ft)
48	23.38	930.8	23.13	23.62	0.120	2.805	2.688	910.68	1.060	1.50	1.53	0.50	331.76	1.53	1.00	331.76	3		>0.6	>5.0	0.945	1.322	Non Liquefiable	0.28	-
49	23.87	930.1	23.62	24.11	0.120	2.864	2.718	878.32	1.034	1.50	1.55	0.50	358.46	1.55	1.00	358.46	4		>0.6	>5.0	0.944	1.348	Non Liquefiable	0.28	-
50	24.36	929.6	24.11	24.61	0.120	2.923	2.745	873.98	1.054	1.50	1.55	0.50	352.58	1.55	1.00	352.58	4		>0.6	>5.0	0.943	1.375	Non Liquefiable	0.28	-
51	24.85	929.1	24.61	25.10	0.120	2.983	2.773	824.34	0.612	1.42	1.36	0.50	340.46	1.36	1.00	340.46	1		>0.6	>5.0	0.942	1.401	Non Liquefiable	0.28	-
52	25.35	928.7	25.10	25.59	0.120	3.041	2.801	790.52	0.577	1.39	1.34	0.50	324.01	1.34	1.00	324.01	1		>0.6	>5.0	0.941	1.427	Non Liquefiable	0.28	-
53	25.84	928.2	25.59	26.08	0.120	3.100	2.830	786.68	0.800	1.48	1.43	0.50	321.41	1.43	1.00	321.41	2		>0.6	>5.0	0.940	1.452	Non Liquefiable	0.28	-
54	26.33	927.7	26.08	26.57	0.120	3.159	2.858	805.67	0.502	1.34	1.30	0.50	300.75	1.30	1.00	300.75	0		>0.6	>5.0	0.939	1.478	Non Liquefiable	0.28	-
55	26.82	927.2	26.57	27.07	0.120	3.218	2.885	830.67	0.577	1.35	1.31	0.50	304.33	1.31	1.00	304.33	0		>0.6	>5.0	0.937	1.504	Non Liquefiable	0.28	-
56	27.32	926.7	27.07	27.56	0.120	3.278	2.915	871.30	0.592	1.41	1.36	0.50	350.62	1.36	1.00	350.62	1		>0.6	>5.0	0.936	1.530	Non Liquefiable	0.28	-
57	27.81	926.2	27.56	28.05	0.120	3.337	2.943	846.96	0.522	1.30	1.30	0.50	339.06	1.30	1.00	339.06	1		>0.6	>5.0	0.935	1.556	Non Liquefiable	0.28	-
58	28.30	925.7	28.05	28.54	0.120	3.395	2.971	887.40	0.341	1.29	1.19	0.50	359.85	1.19	1.00	359.85	0		>0.6	>5.0	0.934	1.581	Non Liquefiable	0.28	-
59	28.79	925.2	28.54	29.04	0.120	3.455	3.000	888.70	0.341	1.25	1.19	0.50	352.37	1.19	1.00	352.37	0		>0.6	>5.0	0.933	1.607	Non Liquefiable	0.28	-
60	29.29	924.7	29.04	29.53	0.120	3.514	3.028	800.10	0.231	1.17	1.11	0.50	330.71	1.11	1.00	330.71	0		>0.6	>5.0	0.932	1.532	Non Liquefiable	0.28	-
61	29.78	924.2	29.53	30.02	0.120	3.573	3.057	831.72	0.474	1.50	1.54	0.50	329.32	1.54	1.00	329.32	3		>0.6	>5.0	0.931	1.558	Non Liquefiable	0.28	-
62	30.27	923.7	30.02	30.51	0.120	3.632	3.085	252.00	1.107	2.03	1.98	0.50	98.81	1.98	1.25	123.74	12		0.250	1.700	0.930	1.580	0.81	0.34	0.49
63	30.76	923.2	30.51	31.00	0.120	3.691	3.113	56.58	2.280	2.74	2.74	1.00	18.88	2.74	4.33	Fine Grained	45		>0.6	>5.0	0.929	1.700	Non Liquefiable	0.34	-
64	31.25	922.8	31.00	31.50	0.120	3.750	3.142	26.30	6.010	3.29	3.29	1.00	7.18	3.29	10.59	Fine Grained	60		>0.6	>5.0	0.928	1.719	Non Liquefiable	0.34	-
65	31.75	922.3	31.5	31.99	0.120	3.809	3.170	71.55	1.163	2.50	2.41	0.70	27.67	2.41	3.04	84.12	27		0.105	0.333	0.918	1.739	0.20	0.45	0.49
66	32.24	921.8	31.99	32.48	0.120	3.868	3.198	37.46	3.211	3.00	3.00	1.00	10.54	3.00	6.70	Fine Grained	58		>0.6	>5.0	0.912	1.758	Non Liquefiable	0.45	-
67	32.73	921.3	32.48	32.97	0.120	3.927	3.227	60.64	1.411	2.61	2.61	1.00	17.55	2.61	3.41	Fine Grained	30		>0.6	>5.0	0.904	1.777	Non Liquefiable	0.48	-
68	33.22	920.8	32.97	33.46	0.120	3.985	3.255	94.47	1.290	2.57	2.47	0.50	24.54	2.47	2.62	64.34	20		0.105	0.341	0.904	1.790	0.25	0.61	0.49
69	33.71	920.3	33.46	33.96	0.120	4.045	3.283	44.10	1.874	2.81	2.81	1.00	12.20	2.81	4.91	Fine-Grained	47		>0.6	>5.0	0.900	1.814	Non Liquefiable	0.61	-
70	34.21	919.8	33.96	34.45	0.120	4.105	3.312	103.66	1.563	2.40	2.35	0.50	34.29	2.35	2.13	42.55	25		0.134	0.445	0.896	1.833	0.32	0.73	0.49
71	34.70	919.3	34.45	34.94	0.120	4.163	3.340	344.04	0.535	1.73	1.65	0.50	129.38	1.65	1.00	129.50	5		0.207	0.060	0.892	1.851	0.66	0.80	0.49
72	35.19	918.8	34.94	35.43	0.120	4.222	3.368	486.40	0.978	1.76	1.68	0.50	186.64	1.68	1.03	191.71	6		>0.6	>5.0	0.888	1.869	Non Liquefiable	0.80	-
73	35.68	918.3	35.43	35.93	0.120	4.282	3.397	719.97	0.980	1.67	1.60	0.50	267.05	1.60	1.00	267.35	4		>0.6	>5.0	0.884	1.888	Non Liquefiable	0.80	-
74	36.18	917.8	35.93	36.42	0.120	4.341	3.425	806.06	0.935	1.62	1.55	0.50	299.34	1.55	1.00	299.34	4		>0.6	>5.0	0.880	1.906	Non Liquefiable	0.80	-
75	36.67	917.3	36.42	36.91	0.120	4.400	3.454	545.06	0.515	1.63	1.54	0.50	201.68	1.54	1.00	201.66	2		>0.6	>5.0	0.875	1.924	Non Liquefiable	0.80	-
76	37.16	916.8	36.91	37.40	0.120	4.459	3.482	420.52	0.716	1.76	1.67	0.50	154.69	1.67	1.02	157.91	0		0.445	1.651	0.872	1.937	1.04	0.81	0.49
77	37.65	916.4	37.40	37.89	0.120	4.517	3.510	401.48	0.676	1.72	1.63	0.50	147.23	1.63	1.00	147.20	5		0.377	1.324	0.868	1.954	0.86	0.84	0.49
78	38.14	915.9	37.89	38.39	0.120	4.577	3.539	253.90	1.539	2.15	2.00	0.50	92.73	2.05	1.38	127.93	15		0.277	0.979	0.864	1.971	0.65	0.94	0.5
79	38.64	915.4	38.39	38.88	0.120	4.636	3.567	441.00	1.223	1.90	1.82	0.50	160.45	1.82	1.12	179.86	9		>0.6	>5.0	0.860	1.987	Non Liquefiable	0.94	-
80	39.13	914.9	38.88	39.37	0.120	4.695	3.595	667.44	0.687	1.71	1.63	0.50	242.00	1.63	1.00	242.00	5		>0.6	>5.0	0.855	2.003	Non Liquefiable	0.94	-
81	39.62	914.4	39.37	39.86	0.120	4.754	3.623	861.64	1.020	1.66	1.57	0.50	311.10	1.57	1.00	311.10	4		>0.6	>5.0	0.852	2.018	Non Liquefiable	0.94	-
82	40.11	913.9	39.86	40.35	0.120	4.813	3.652	967.70	0.894	1.57	1.49	0.50	348.05	1.49	1.00	348.05	3		>0.6	>5.0	0.848	2.034	Non Liquefiable	0.94	-
83	40.60	913.4	40.35	40.85	0.120	4.872	3.680	905.96	1.015	1.64	1.56	0.50	324.94	1.56	1.00	324.94	4		>0.6	>5.0	0.844	2.049	Non Liquefiable	0.94	-
84	41.10	912.9	40.85	41.34	0.120	4.931	3.709	824.80	0.507	1.45	1.38	0.50	294.37	1.38	1.00	294.37	1		>0.6	>5.0	0.840	2.064	Non Liquefiable	0.94	-
85	41.59	912.4	41.34	41.83	0.120	4.990	3.737	752.52	0.775	1.61	1.52	0.50	267.19	1.52	1.00	267.19	3		>0.6	>5.0	0.836	2.079	Non Liquefiable	0.94	-
86	42.08	911.9	41.83	42.32	0.120	5.049	3.765	895.72	0.593	1.40	1.30	0.50	317.27	1.30	1.00	317.27	1		>0.6	>5.0	0.832	2.093	Non Liquefiable	0.94	-
87	42.57	911.4	42.32	42.81	0.120	5.108	3.793	857.52	0.724	1.56	1.47	0.50	302.80	1.47	1.00	302.80	2		>0.6	>5.0	0.828	2.107	Non Liquefiable	0.94	-
88	43.06	910.9	42.81	43.31	0.120	5.167	3.822	524.87	0.835	1.61	1.53	0.50	375.06	1.53	1.00	375.06	3		>0.6	>5.0	0.824	2.121	Non Liquefiable	0.94	-
89	43.56	910.4	43.31	43.80	0.120	5.227	3.850	1046.77	0.824	1.50	1.45	0.50	366.45	1.40	1.00	366.45	2		>0.6	>5.0	0.820	2.135	Non Liquefiable	0.94	-
90	44.05	910.0	43.80	44.29	0.120	5.285	3.870	1049.58	1.307	1.71	1.63	0.50	377.60	1.53	1.00	377.60	5		>0.6	>5.0	0.815	2.148	Non Liquefiable	0.94	
91	44.54	909.5	44.29	44.78	0.120	5.344	3.907	1107.96	1.176	1.64	1.56	0.50	394.96	1.54	1.00	394.96	4		>0.6	>5.0	0.812	2.162	Non Liquefiable	0.94	-
92	45.03	909.0	44.78	45.28	0.120	5.404	3.935	1030.29	0.794	1.54	1.45	0.50	356.65	1.45	1.00	356.56	2		>0.6	>5.0	0.808	2.175	Non Liquefiable	0.94	-
93	45.53	908.5	45.28	45.77	0.120	5.463	3.964	923.18	0.826	1.56	1.49	0.50	318.69	1.49	1.00	318.69	3		>0.6	>5.0	0.804	2.188	Non Liquefiable	0.94	-
94	46.02	908.0	45.77	46.26	0.120	5.522	3.992	792.12	0.634	1.54	1.44	0.50	272.47	1.44	1.00	272.47	2		>0.6	>5.0	0.800	2.201	Non Liquefiable	0.94	-
95	46.51	907.5	46.26	46.75	0.120	5.581	4.020	710.12	0.504	1.56	1.45	0.50	245.47	1.45	1.00	245.47	2		>0.6	>5.0	0.796	2.213	Non Liquefiable	0.94	-
96	47.00	907.0	46.75	47.24	0.120	5.639	4.049	766.12	0.675	1.56	1.48	0.50	258.27	1.48	1.00	258.77	3		>0.6	>5.0	0.792	2.225	Non Liquefiable	0.94	-
97	47.49	906.5	47.24	47.74	0.120	5.699	4.077	888.14	1.449	1.78	1.70	0.50	305.37	1.70	1.00	315.91	8		>0.6	>5.0	0.788	2.237	Non Liquefiable	0.94	-
98	47.99	906.0	47.74	48.23	0.120	5.758	4.106	988.44	0.493	1.41	1.30	0.50	334.60	1.30	1.00	334.60	0		>0.6	>5.0	0.783	2.249	Non Liquefiable	0.94	-
99	48.48	905.5	48.23	48.72	0.120	5.817	4.134	849.50	0.449	1.45	1.33	0.50	287.18	1.33	1.00	287.18	1		>0.6	>5.0	0.780	2.261	Non Liquefiable	0.94	-
100	48.97	905.0	48.72	49.21	0.120	5.876	4.162	[illegible]	3.403	2.45	2.35	0.50	47.81	2.35	2.11	174.35	24		>0.6	>5.0	0.776	2.272	Non Liquefiable	0.94	-
101	49.46	904.5	49.21	49.70	0.120	5.935	4.191	114.72	9.401	3.01	3.01	1.00	25.90	3.01	6.84	Fine Grained	59		>0.6	>5.0	0.772	2.283	Non Liquefiable	0.94	-

Total Thickness of Potentially Liquefiable Zones Remaining = 10.3

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*******************************************
*                                         *
*        FRISKSP - IBM-PC VERSION         *
*                                         *
*  Modified from *FRISK* (McGuire 1978)   *
*   To Perform Probabilistic Earthquake   *
*  Hazard Analyses Using Multiple Forms   *
* of Ground-Motion-Attenuation Relations  *
*                                         *
*   Modifications by:  Thomas F. Blake    *
*              - 1988-2000 -              *
*                                         *
*              VERSION 4.00               *
*             (Visual Fortran)            *
*******************************************
```

TITLE: LandMark Village - Boore et al (1997) Site Class D

IPR_FILE
 0

IPLOT
 0

SITE CONDITION
 0.00

BASEMENT DEPTH (km)
 5.00

RHGA FACTOR RHGA DIST (km)
 1.000 0.000

NFLT NSITE NPROB NATT LCD

21 1 2 6 1

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
C11	C12	C13	C14							
1	-0.3130	0.5270	0.0000	-0.7780	-0.3710	1396.0000	5.5700	250.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000							
ATT	C15	C16	C17	C18	C19	C20	C21	C22	C23	PER
DSMIN	SIGA	IRELAF	ICHK							
1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.5200	35	0							

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
C11	C12	C13	C14							
2	-0.1170	0.5270	0.0000	-0.7780	-0.3710	1396.0000	5.5700	250.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000							
ATT	C15	C16	C17	C18	C19	C20	C21	C22	C23	PER
DSMIN	SIGA	IRELAF	ICHK							
2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.5200	35	0							

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
C11	C12	C13	C14							
3	-0.3130	0.5270	0.0000	-0.7780	-0.3710	1396.0000	5.5700	250.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000							
ATT	C15	C16	C17	C18	C19	C20	C21	C22	C23	PER
DSMIN	SIGA	IRELAF	ICHK							
3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.5200	35	0							

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
C11	C12	C13	C14							
4	-0.1170	0.5270	0.0000	-0.7780	-0.3710	1396.0000	5.5700	250.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000							
ATT	C15	C16	C17	C18	C19	C20	C21	C22	C23	PER
DSMIN	SIGA	IRELAF	ICHK							
4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.5200	35	0							

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10
C11	C12	C13	C14							
5	-0.1170	0.5270	0.0000	-0.7780	-0.3710	1396.0000	5.5700	250.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000							
ATT	C15	C16	C17	C18	C19	C20	C21	C22	C23	PER
DSMIN	SIGA	IRELAF	ICHK							
5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.5200	35	0							

ATT	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10

```
C11        C12        C13        C14
  6    -0.1170     0.5270     0.0000    -0.7780    -0.3710 1396.0000     5.5700  250.0000     0.0000     0.0000
0.0000     0.0000     0.0000     0.0000
ATT        C15        C16        C17        C18        C19        C20        C21        C22        C23        PER
DSMIN       SIGA     IRELAF      ICHK
  6     0.0000     0.0000     0.0000     0.0000     0.0000     0.0000     0.0000     0.0000     0.0000     0.0000
0.0000     0.5200         35          0

 PROBLEM DATA:

 BOORE ET AL(1997) NEHRP D (250)1 AMPLITUDES:
     15          0.100     0.200     0.300     0.400     0.500     0.600     0.700     0.800     0.900     1.000
                 1.100     1.200     1.300     1.400     1.500

      MAGNITUDE WEIGHTING FACTORS:      MWF:    0     MWF MAGNITUDE:       0.00

 BOORE ET AL(1997) NEHRP D (250)2 AMPLITUDES:
     15          0.100     0.200     0.300     0.400     0.500     0.600     0.700     0.800     0.900     1.000
                 1.100     1.200     1.300     1.400     1.500

      MAGNITUDE WEIGHTING FACTORS:      MWF:    2     MWF MAGNITUDE:       7.50

 RISKS SPECIFIED:
   5      0.010000  0.005000  0.002105  0.001000  0.000404

 SITE COORDINATES:
  1     -118.6386   34.4243


 FAULT INFORMATION:

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 FAULT  1

 FAULT NAME:  HOLSER


 NFP     NRL    ATTENUATION CODES:
  5      10       2  4

 AMMIN    AMSTEP   IRATE     RATE    BETA    ECTR    ECDP    COEF
 5.000    0.1000       1   0.4000   2.072   1.000   2.000   1.000

 NMAX AMMAX   PMAX
  1    6.50   1.00
```

```
  dmchar ampchar dmpchar
    0.50    6.00    1.00

 Slip Rate (  0.4000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.280E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  6.5000  1.0000   0.00212   0.00097   0.00115


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1     -118.7533   34.4386
  2     -118.7345   34.4386
  3     -118.6741   34.4499
  4     -118.6427   34.4487
  5     -118.5483   34.4172

 NDP
   2
 ORIGINAL FAULT CROSS SECTION
  1        0.0000    0.0000
  2        5.9000   12.7000

 Computed Total Fault Area = 0.26E+03

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 FAULT  2

 FAULT NAME:  NORTHRIDGE (E. Oak Ridge)


 NFP     NRL    ATTENUATION CODES:
  2      10       5  6

 AMMIN    AMSTEP  IRATE    RATE    BETA    ECTR    ECDP    COEF
 5.000    0.1000      1  1.5000  2.072   1.500   2.000   1.000

 NMAX AMMAX  PMAX
  1    6.90  1.00
```

```
  dmchar ampchar dmpchar
    0.50    6.40    1.00

 Slip Rate (  1.5000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.360E+12
 Input Fault Area    - cm**2
0.682E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  6.9000  1.0000   0.00891   0.00602   0.00288


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA    SIG_RA  -3.490     0.910     0.240

 FAULT SEGMENT COORDINATES
  1    -118.7027   34.4057
  2    -118.4078   34.2781

 NDP
    3
 ORIGINAL FAULT CROSS SECTION
  1       0.0000    4.9900
  2       0.0000    5.0000
  3      16.3000   19.7000

 Computed Total Fault Area = 0.67E+03

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 FAULT  3

 FAULT NAME:  SAN GABRIEL


 NFP    NRL   ATTENUATION CODES:
 10     10      1  3

 AMMIN   AMSTEP  IRATE    RATE   BETA   ECTR   ECDP   COEF
 5.000   0.1000      1  1.0000  2.072  3.600  2.000  1.000

 NMAX AMMAX  PMAX
  1    7.00  1.00
```

```
   dmchar ampchar dmpchar
     0.50    6.50    1.00

 Slip Rate (  1.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.936E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  7.0000  1.0000   0.00616   0.00445   0.00171


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1     -118.2802   34.3179
  2     -118.3118   34.3394
  3     -118.3904   34.3598
  4     -118.4577   34.3853
  5     -118.5587   34.4363
  6     -118.5975   34.4649
  7     -118.6873   34.5536
  8     -118.7026   34.5700
  9     -118.7312   34.5792
 10     -118.8761   34.7139

 NDP
    2
 ORIGINAL FAULT CROSS SECTION
  1        0.0000    0.0000
  2        0.0000   13.0000

 Computed Total Fault Area = 0.93E+03

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 FAULT  4

 FAULT NAME:  OAK RIDGE (Onshore)


 NFP    NRL   ATTENUATION CODES:
```

```
 9      10       2  4

AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000      1   4.0000   2.072   2.500   2.000   1.000

NMAX AMMAX  PMAX
 1    6.90  1.00


  dmchar ampchar dmpchar
    0.50    6.40    1.00

 Slip Rate (  4.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.700E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX      ARATE = EX-RATE + CH-RATE
      1  6.9000  1.0000   0.02235   0.01511   0.00723


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1    -119.2050   34.2481
  2    -119.1582   34.2630
  3    -119.0974   34.3165
  4    -119.0402   34.3522
  5    -118.9589   34.3631
  6    -118.8805   34.3813
  7    -118.8104   34.3850
  8    -118.7742   34.4013
  9    -118.7227   34.3978

 NDP
   3
 ORIGINAL FAULT CROSS SECTION
  1       0.0000    1.0000
  2       0.0000    1.1000
  3       5.9000   13.7000

 Computed Total Fault Area = 0.67E+03

 ------------------------------------------------------------------------------
```

FAULT 5

FAULT NAME: SANTA SUSANA

NFP	NRL	ATTENUATION CODES:	
8	10	2	4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	5.0000	2.072	1.300	2.000	1.000

NMAX	AMMAX	PMAX
1	6.60	1.00

dmchar	ampchar	dmpchar
0.50	6.10	1.00

Slip Rate (5.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area - cm**2
0.432E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.6000	1.0000	0.03249	0.01677	0.01573

IND_RL
2

RUPTURE AREA VS. MAGNITUDE	A_RA	B_RA	SIG_RA
	-3.490	0.910	0.240

FAULT SEGMENT COORDINATES

1	-118.4950	34.3242
2	-118.4955	34.3242
3	-118.5340	34.3030
4	-118.5811	34.3204
5	-118.6163	34.3229
6	-118.6339	34.3330
7	-118.7081	34.3506
8	-118.7672	34.3594

NDP
2
ORIGINAL FAULT CROSS SECTION

1	0.0000	0.0000

```
 2       9.2000   13.1000

Computed Total Fault Area = 0.41E+03

--------------------------------------------------------------------------------

FAULT  6

FAULT NAME:  SAN CAYETANO


NFP    NRL   ATTENUATION CODES:
 9     10      2  4

AMMIN   AMSTEP  IRATE    RATE   BETA   ECTR   ECDP   COEF
5.000   0.1000      1  6.0000  2.072  2.200  2.000  1.000

NMAX AMMAX  PMAX
 1    6.80  1.00


  dmchar ampchar dmpchar
    0.50    6.30    1.00

Slip Rate (  6.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area    - cm**2
0.660E+13
LOG10[Mo(m)]        = (1.50)m + (16.05)
  IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
     1  6.8000  1.0000   0.03867   0.02422   0.01445


IND_RL
     2

RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA    SIG_RA  -3.490     0.910     0.240

FAULT SEGMENT COORDINATES
 1    -118.7621   34.4361
 2    -118.8313   34.4047
 3    -118.9130   34.4172
 4    -118.9281   34.4587
 5    -118.9382   34.4612
 6    -118.9835   34.4348
 7    -119.0690   34.4361
 8    -119.1067   34.4386
```

```
 9    -119.1708   34.4625

NDP
  2
ORIGINAL FAULT CROSS SECTION
 1       0.0000    0.0000
 2       7.5000   13.0000

Computed Total Fault Area = 0.60E+03

--------------------------------------------------------------------------------

FAULT  7

FAULT NAME:  SIERRA MADRE (San Fernando)


NFP     NRL    ATTENUATION CODES:
 8      10       2  4

AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000      1  2.0000   2.072   0.900   2.000   1.000

NMAX AMMAX  PMAX
 1    6.70  1.00


   dmchar ampchar dmpchar
     0.50    6.20    1.00

 Slip Rate (  2.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.324E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX     ARATE = EX-RATE + CH-RATE
      1  6.7000   1.0000    0.00781   0.00447   0.00334


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1    -118.2940   34.2782
  2    -118.2951   34.2782
```

```
 3     -118.3196    34.2745
 4     -118.3956    34.2905
 5     -118.4189    34.3039
 6     -118.4520    34.2940
 7     -118.4778    34.3027
 8     -118.4790    34.3027

NDP
  2
ORIGINAL FAULT CROSS SECTION
 1        0.0000     0.0000
 2       12.7000    12.7000

Computed Total Fault Area = 0.33E+03

--------------------------------------------------------------------------------

FAULT  8

FAULT NAME:  SIMI-SANTA ROSA


NFP     NRL    ATTENUATION CODES:
 5      10       2  4

AMMIN    AMSTEP   IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000       1   1.0000   2.072   1.500   2.000   1.000

NMAX AMMAX   PMAX
 1    6.70   1.00


   dmchar ampchar dmpchar
     0.50    6.20    1.00

 Slip Rate (  1.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.450E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX      ARATE = EX-RATE + CH-RATE
      1  6.7000   1.0000    0.00543   0.00311   0.00232


 IND_RL
      2
```

RUPTURE AREA VS. MAGNITUDE A_RA B_RA SIG_RA -3.490 0.910 0.240

FAULT SEGMENT COORDINATES

1	-118.7982	34.2901
2	-118.9084	34.2578
3	-118.9364	34.2615
4	-118.9680	34.2615
5	-119.1147	34.2261

NDP
3

ORIGINAL FAULT CROSS SECTION

1	0.0000	1.0000
2	0.0000	1.1000
3	7.5000	14.0000

Computed Total Fault Area = 0.45E+03

FAULT 9

FAULT NAME: VERDUGO

NFP	NRL	ATTENUATION CODES:
7	10	2 4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	0.5000	2.072	1.400	2.000	1.000

NMAX	AMMAX	PMAX
1	6.70	1.00

dmchar	ampchar	dmpchar
0.50	6.20	1.00

Slip Rate (0.5000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area - cm**2
0.522E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.7000	1.0000	0.00315	0.00180	0.00135

```
IND_RL
     2

RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

FAULT SEGMENT COORDINATES
 1     -118.1536    34.1313
 2     -118.1865    34.1496
 3     -118.2285    34.1551
 4     -118.2907    34.1971
 5     -118.3657    34.2227
 6     -118.4077    34.2538
 7     -118.4206    34.2612

NDP
  2
ORIGINAL FAULT CROSS SECTION
 1       0.0000     0.0000
 2      12.7000    12.7000

Computed Total Fault Area = 0.52E+03

--------------------------------------------------------------------------------

FAULT 10

FAULT NAME:  SANTA YNEZ (East)


NFP     NRL    ATTENUATION CODES:
 6      10       1  3

AMMIN    AMSTEP   IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000       1   2.0000   2.072   3.400   2.000   1.000

NMAX AMMAX   PMAX
 1    7.00   1.00


   dmchar ampchar dmpchar
     0.50    6.50    1.00

 Slip Rate (  2.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.884E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
```

IMAX	AMMAX	PMAX	ARATE =	EX-RATE +	CH-RATE
1	7.0000	1.0000	0.01164	0.00840	0.00323

IND_RL
2

RUPTURE AREA VS. MAGNITUDE	A_RA	B_RA	SIG_RA	-3.490	0.910	0.240

FAULT SEGMENT COORDINATES

1	-119.6298	34.4939
2	-119.5091	34.4864
3	-119.3783	34.5065
4	-119.1972	34.5367
5	-119.1167	34.5317
6	-118.9055	34.5870

NDP
2

ORIGINAL FAULT CROSS SECTION

1	0.0000	0.0000
2	2.3000	12.8000

Computed Total Fault Area = 0.89E+03

FAULT 11

FAULT NAME: SAN ANDREAS - 1857 Rupture

NFP	NRL	ATTENUATION CODES:
12	10	1 3

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	34.0000	2.072	17.200	2.000	0.500

NMAX	AMMAX	PMAX
1	7.80	1.00

dmchar	ampchar	dmpchar
0.50	7.30	1.00

Slip Rate (34.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.300E+12

```
 Input Fault Area     - cm**2
0.414E+14
 LOG10[Mo(m)]          = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX      ARATE = EX-RATE + CH-RATE
      1  7.8000   1.0000    0.22372    0.20895    0.01477


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1     -120.5605    36.0019
  2     -120.2928    35.7488
  3     -119.8632    35.3106
  4     -119.8598    35.3072
  5     -119.6673    35.1336
  6     -119.4061    34.9395
  7     -119.2103    34.8639
  8     -118.9010    34.8175
  9     -118.5075    34.7006
 10     -118.5024    34.6989
 11     -118.0075    34.5116
 12     -117.5298    34.3106

 NDP
   2
 ORIGINAL FAULT CROSS SECTION
  1        0.0000     0.0000
  2        0.0000    12.0000

 Computed Total Fault Area = 0.41E+04

 ------------------------------------------------------------------------------

 FAULT 12

 FAULT NAME:  SAN ANDREAS - Mojave


 NFP     NRL    ATTENUATION CODES:
  3      10       1  3

 AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
 5.000    0.1000      1 30.0000  2.072  4.900  2.000  0.500

 NMAX AMMAX   PMAX
```

```
 1    7.10  1.00


   dmchar ampchar dmpchar
     0.50    6.60    1.00

 Slip Rate ( 30.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.300E+12
 Input Fault Area    - cm**2
0.119E+14
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  7.1000  1.0000   0.17754   0.13551   0.04202


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1    -118.5024   34.6989
  2    -118.0075   34.5116
  3    -117.5298   34.3106

 NDP
   2
 ORIGINAL FAULT CROSS SECTION
  1       0.0000    0.0000
  2       0.0000   12.0000

 Computed Total Fault Area = 0.12E+04

 ------------------------------------------------------------------------------

 FAULT 13

 FAULT NAME:  SAN ANDREAS - Carrizo


 NFP    NRL   ATTENUATION CODES:
  6     10      1  3

 AMMIN   AMSTEP  IRATE    RATE   BETA   ECTR   ECDP   COEF
 5.000   0.1000      1 34.0000  2.072  7.200  2.000  0.500

 NMAX AMMAX   PMAX
```

```
 1    7.20  1.00


   dmchar ampchar dmpchar
     0.50    6.70    1.00

 Slip Rate ( 34.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.300E+12
 Input Fault Area    - cm**2
0.174E+14
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  7.2000  1.0000   0.24626   0.19696   0.04930


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1     -119.8598    35.3072
  2     -119.6673    35.1336
  3     -119.4061    34.9395
  4     -119.2103    34.8639
  5     -118.9010    34.8175
  6     -118.5075    34.7006

 NDP
   2
 ORIGINAL FAULT CROSS SECTION
  1        0.0000     0.0000
  2        0.0000    12.0000

 Computed Total Fault Area = 0.17E+04

 ------------------------------------------------------------------------------

 FAULT 14

 FAULT NAME:  SIERRA MADRE


 NFP     NRL    ATTENUATION CODES:
 12      10       2  4

 AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
```

```
 5.000    0.1000       1  3.0000   2.072   2.800   2.000   1.000

 NMAX AMMAX  PMAX
  1    7.00  1.00


   dmchar ampchar dmpchar
     0.50    6.50    1.00

 Slip Rate (  3.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.103E+14
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  7.0000  1.0000   0.02034   0.01469   0.00565


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

 FAULT SEGMENT COORDINATES
  1     -117.7397   34.1231
  2     -117.7691   34.1317
  3     -117.8176   34.1323
  4     -117.8807   34.1470
  5     -117.9402   34.1501
  6     -118.0027   34.1752
  7     -118.0683   34.1758
  8     -118.1118   34.2010
  9     -118.1492   34.2028
 10     -118.2461   34.2279
 11     -118.2896   34.2751
 12     -118.2960   34.2751

 NDP
   2
 ORIGINAL FAULT CROSS SECTION
  1       0.0000    0.0000
  2      12.7000   12.7000

 Computed Total Fault Area = 0.11E+04

 ------------------------------------------------------------------------
```

FAULT 15

FAULT NAME: MALIBU COAST

NFP	NRL	ATTENUATION CODES:	
4	10	2	4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	0.3000	2.072	1.800	2.000	1.000

NMAX	AMMAX	PMAX
1	6.70	1.00

dmchar	ampchar	dmpchar
0.50	6.20	1.00

Slip Rate (0.3000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area - cm**2
0.481E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.7000	1.0000	0.00174	0.00100	0.00074

IND_RL
2

RUPTURE AREA VS. MAGNITUDE	A_RA	B_RA	SIG_RA	-3.490	0.910	0.240

FAULT SEGMENT COORDINATES

1	-118.5333	34.0299
2	-118.6339	34.0412
3	-118.6666	34.0387
4	-118.9332	34.0513

NDP
2
ORIGINAL FAULT CROSS SECTION

1	0.0000	0.0000
2	3.4000	12.6000

Computed Total Fault Area = 0.48E+03

FAULT 16

FAULT NAME: HOLLYWOOD

NFP	NRL	ATTENUATION CODES:	
4	10	2	4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	1.0000	2.072	0.800	2.000	1.000

NMAX	AMMAX	PMAX
1	6.40	1.00

dmchar	ampchar	dmpchar
0.50	5.90	1.00

Slip Rate (1.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area - cm**2
0.238E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.4000	1.0000	0.00575	0.00229	0.00346

IND_RL
2

RUPTURE AREA VS. MAGNITUDE	A_RA	B_RA	SIG_RA
	-3.490	0.910	0.240

FAULT SEGMENT COORDINATES

1	-118.2302	34.1192
2	-118.3170	34.1104
3	-118.3723	34.0991
4	-118.4063	34.0827

NDP
2
ORIGINAL FAULT CROSS SECTION

1	0.0000	0.0000
2	4.8000	13.2000

Computed Total Fault Area = 0.25E+03

FAULT 17

FAULT NAME: SANTA MONICA

NFP	NRL	ATTENUATION CODES:
3	10	2 4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	1.0000	2.072	1.400	2.000	1.000

NMAX	AMMAX	PMAX
1	6.60	1.00

dmchar	ampchar	dmpchar
0.50	6.10	1.00

Slip Rate (1.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.360E+12
Input Fault Area - cm**2
0.364E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.6000	1.0000	0.00597	0.00308	0.00289

IND_RL
2

RUPTURE AREA VS. MAGNITUDE A_RA B_RA SIG_RA -3.490 0.910 0.240

FAULT SEGMENT COORDINATES

1	-118.4085	34.0814
2	-118.5244	34.0263
3	-118.6855	33.9896

NDP
2
ORIGINAL FAULT CROSS SECTION

1	0.0000	0.0000
2	3.4000	12.6000

Computed Total Fault Area = 0.35E+03

FAULT 18

FAULT NAME: VENTURA - PITAS POINT

NFP	NRL	ATTENUATION CODES:	
6	10	2	4

AMMIN	AMSTEP	IRATE	RATE	BETA	ECTR	ECDP	COEF
5.000	0.1000	1	1.0000	2.072	2.000	2.000	1.000

NMAX	AMMAX	PMAX
1	6.80	1.00

dmchar	ampchar	dmpchar
0.50	6.30	1.00

Slip Rate (1.0000 mm/yr) Converted to Activity Rate:
Input Shear Modulus - dyne/cm**2
0.330E+12
Input Fault Area - cm**2
0.533E+13
LOG10[Mo(m)] = (1.50)m + (16.05)

IMAX	AMMAX	PMAX	ARATE = EX-RATE + CH-RATE		
1	6.8000	1.0000	0.00520	0.00326	0.00195

IND_RL
2

RUPTURE AREA VS. MAGNITUDE A_RA B_RA SIG_RA -3.490 0.910 0.240

FAULT SEGMENT COORDINATES

1	-119.1411	34.3453
2	-119.1940	34.3088
3	-119.2243	34.2928
4	-119.3034	34.2892
5	-119.4040	34.3166
6	-119.5527	34.3093

NDP
3
ORIGINAL FAULT CROSS SECTION

1	0.0000	1.0000
2	0.0000	1.1000

```
 3       3.4000   13.6000

Computed Total Fault Area = 0.51E+03

--------------------------------------------------------------------------------

FAULT 19

FAULT NAME:  M.RIDGE-ARROYO PARIDA-SANTA ANA


NFP     NRL    ATTENUATION CODES:
14      10       2  4

AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000      1  0.4000  2.072   3.200   2.000   1.000

NMAX AMMAX  PMAX
 1    6.70  1.00


  dmchar ampchar dmpchar
    0.50    6.20    1.00

 Slip Rate (  0.4000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.975E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX      ARATE = EX-RATE + CH-RATE
      1  6.7000  1.0000   0.00470   0.00269   0.00201


 IND_RL
      2

 RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA    SIG_RA  -3.490     0.910     0.240

 FAULT SEGMENT COORDINATES
  1    -119.1658   34.4701
  2    -119.2878   34.4298
  3    -119.3456   34.4235
  4    -119.4261   34.4160
  5    -119.4714   34.4273
  6    -119.5104   34.4311
  7    -119.6110   34.4411
  8    -119.6914   34.4499
```

```
 9    -119.7430   34.4424
10    -119.7506   34.4411
11    -119.7857   34.4336
12    -119.8071   34.4336
13    -119.8688   34.4248
14    -119.8977   34.4298

NDP
  2
ORIGINAL FAULT CROSS SECTION
 1       0.0000    0.0000
 2       7.5000   13.0000

Computed Total Fault Area = 0.10E+04

--------------------------------------------------------------------------------

FAULT 20

FAULT NAME:  ANACAPA-DUME


NFP    NRL   ATTENUATION CODES:
 6     10      2  4

AMMIN   AMSTEP  IRATE    RATE   BETA   ECTR   ECDP   COEF
5.000   0.1000      1  3.0000  2.072  3.700  2.000  1.000

NMAX AMMAX  PMAX
 1    7.30  1.00


  dmchar ampchar dmpchar
    0.50    6.80    1.00

 Slip Rate (  3.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.210E+14
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX   AMMAX    PMAX     ARATE = EX-RATE + CH-RATE
      1  7.3000  1.0000   0.02430   0.02021   0.00409


 IND_RL
      2
```

```
RUPTURE AREA VS. MAGNITUDE  A_RA      B_RA     SIG_RA  -3.490      0.910      0.240

FAULT SEGMENT COORDINATES
 1     -118.6945    33.9849
 2     -118.7913    33.9530
 3     -118.9120    33.9508
 4     -119.1410    33.9838
 5     -119.3403    33.9792
 6     -119.4963    33.9883

NDP
  2
ORIGINAL FAULT CROSS SECTION
 1       0.0000     0.0000
 2      19.8000    19.8000

Computed Total Fault Area = 0.20E+04

--------------------------------------------------------------------------------

FAULT 21

FAULT NAME:  NEWPORT-INGLEWOOD (L.A.Basin)


NFP     NRL    ATTENUATION CODES:
 5      10       1  3

AMMIN    AMSTEP  IRATE     RATE    BETA    ECTR    ECDP    COEF
5.000    0.1000      1  1.0000   2.072   3.200   2.000   1.000

NMAX AMMAX   PMAX
 1    6.90   1.00


   dmchar ampchar dmpchar
     0.50    6.40    1.00

 Slip Rate (  1.0000 mm/yr) Converted to Activity Rate:
 Input Shear Modulus - dyne/cm**2
0.330E+12
 Input Fault Area    - cm**2
0.832E+13
 LOG10[Mo(m)]        = (1.50)m + (16.05)
   IMAX    AMMAX     PMAX      ARATE = EX-RATE + CH-RATE
      1  6.9000  1.0000   0.00664   0.00449   0.00215
```

```
IND_RL
     2

RUPTURE AREA VS. MAGNITUDE  A_RA       B_RA     SIG_RA  -3.490      0.910      0.240

FAULT SEGMENT COORDINATES
 1     -118.3723    34.0337
 2     -118.1862    33.8073
 3     -118.1510    33.7822
 4     -118.1208    33.7746
 5     -117.9246    33.6061

NDP
   2
ORIGINAL FAULT CROSS SECTION
 1        0.0000     0.0000
 2        0.0000    13.0000

Computed Total Fault Area = 0.83E+03

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 SITE 1 COORDINATES: -118.6386  34.4243

 BOORE ET AL(1997) NEHRP D (250)1
      AMPLITUDES (g):
0.1000E+000.2000E+000.3000E+000.4000E+000.5000E+000.6000E+000.7000E+000.8000E+000.9000E+000.1000E+01
      LN (AMPLITUDE):       -2.30     -1.61     -1.20     -0.92     -0.69     -0.51     -0.36     -0.22
-0.11      0.00
   FAULT    1 E(NO/YR)
0.2096E-020.1835E-020.1429E-020.1046E-020.7423E-030.5192E-030.3612E-030.2514E-030.1756E-030.1233E-03
   FAULT    2 E(NO/YR)
0.7261E-020.4503E-020.2916E-020.1941E-020.1304E-020.8798E-030.5967E-030.4074E-030.2802E-030.1943E-03
   FAULT    3 E(NO/YR)
0.4270E-020.2436E-020.1491E-020.9303E-030.5836E-030.3680E-030.2339E-030.1501E-030.9735E-040.6388E-04
   FAULT    4 E(NO/YR)
0.1342E-010.6350E-020.3254E-020.1752E-020.9773E-030.5616E-030.3311E-030.1998E-030.1231E-030.7731E-04
   FAULT    5 E(NO/YR)
0.3130E-010.2544E-010.1903E-010.1378E-010.9808E-020.6921E-020.4869E-020.3429E-020.2423E-020.1720E-02
   FAULT    6 E(NO/YR)
0.2448E-010.1084E-010.4953E-020.2358E-020.1169E-020.6021E-030.3212E-030.1769E-030.1002E-030.5828E-04
   FAULT    7 E(NO/YR)
0.5602E-020.2581E-020.1143E-020.5135E-030.2380E-030.1144E-030.5706E-040.2945E-040.1569E-040.8602E-05
   FAULT    8 E(NO/YR)
0.3152E-020.1138E-020.4131E-030.1580E-030.6418E-040.2762E-040.1252E-040.5948E-050.2944E-050.1511E-05
   FAULT    9 E(NO/YR)
```

0.1477E-020.4019E-030.1151E-030.3625E-040.1252E-040.4693E-050.1887E-050.8056E-060.3624E-060.1707E-06
FAULT 10 E(NO/YR)
0.2786E-020.5931E-030.1496E-030.4321E-040.1397E-040.4963E-050.1907E-050.7832E-060.3405E-060.1555E-06
FAULT 11 E(NO/YR)
0.1320E-010.3970E-020.1530E-020.6243E-030.2670E-030.1196E-030.5601E-040.2731E-040.1381E-040.7223E-05
FAULT 12 E(NO/YR)
0.1959E-010.4129E-020.1037E-020.2973E-030.9531E-040.3355E-040.1278E-040.5200E-050.2241E-050.1015E-05
FAULT 13 E(NO/YR)
0.2047E-010.4221E-020.1079E-020.3169E-030.1041E-030.3746E-040.1456E-040.6035E-050.2646E-050.1217E-05
FAULT 14 E(NO/YR)
0.5994E-020.1403E-020.3847E-030.1192E-030.4093E-040.1532E-040.6163E-050.2637E-050.1190E-050.5622E-06
FAULT 15 E(NO/YR)
0.7029E-030.1489E-030.3453E-040.9145E-050.2731E-050.9025E-060.3249E-060.1257E-060.5174E-070.2247E-07
FAULT 16 E(NO/YR)
0.2275E-020.3994E-030.7818E-040.1807E-040.4826E-050.1453E-050.4824E-060.1739E-060.6724E-070.2760E-07
FAULT 17 E(NO/YR)
0.2328E-020.4461E-030.9489E-040.2346E-040.6624E-050.2088E-050.7220E-060.2696E-060.1076E-060.4541E-07
FAULT 18 E(NO/YR)
0.1298E-020.2083E-030.4044E-040.9409E-050.2539E-050.7729E-060.2596E-060.9464E-070.3698E-070.1533E-07
FAULT 19 E(NO/YR)
0.8252E-030.9859E-040.1589E-040.3249E-050.7960E-060.2244E-060.7073E-070.2444E-070.9113E-080.3626E-08
FAULT 20 E(NO/YR)
0.6369E-020.1683E-020.5410E-030.1908E-030.7268E-040.2963E-040.1282E-040.5840E-050.2785E-050.1382E-05
FAULT 21 E(NO/YR)
0.8024E-030.8572E-040.1274E-040.2437E-050.5642E-060.1514E-060.4570E-070.1519E-070.5467E-080.2107E-08
TOTAL E(NO/YR)
0.1697E+000.7291E-010.3974E-010.2417E-010.1551E-010.1024E-010.6891E-020.4699E-020.3241E-020.2259E-02
TOTAL RISK
0.1561E+000.7031E-010.3896E-010.2388E-010.1539E-010.1019E-010.6867E-020.4688E-020.3236E-020.2257E-02

AMPLITUDES (g):	0.1100E+01	0.1200E+01	0.1300E+01	0.1400E+01	0.1500E+01
LN (AMPLITUDE):	0.10	0.18	0.26	0.34	0.41
FAULT 1 E(NO/YR)	0.8713E-04	0.6200E-04	0.4444E-04	0.3209E-04	0.2334E-04
FAULT 2 E(NO/YR)	0.1359E-03	0.9586E-04	0.6819E-04	0.4891E-04	0.3537E-04
FAULT 3 E(NO/YR)	0.4241E-04	0.2848E-04	0.1934E-04	0.1327E-04	0.9201E-05
FAULT 4 E(NO/YR)	0.4942E-04	0.3211E-04	0.2118E-04	0.1416E-04	0.9591E-05
FAULT 5 E(NO/YR)	0.1229E-02	0.8832E-03	0.6392E-03	0.4657E-03	0.3417E-03
FAULT 6 E(NO/YR)	0.3470E-04	0.2110E-04	0.1309E-04	0.8264E-05	0.5303E-05
FAULT 7 E(NO/YR)	0.4844E-05	0.2795E-05	0.1648E-05	0.9923E-06	0.6085E-06
FAULT 8 E(NO/YR)	0.8017E-06	0.4379E-06	0.2456E-06	0.1411E-06	0.8287E-07
FAULT 9 E(NO/YR)	0.8367E-07	0.4250E-07	0.2229E-07	0.1203E-07	0.6663E-08
FAULT 10 E(NO/YR)	0.7414E-07	0.3672E-07	0.1882E-07	0.9937E-08	0.5394E-08
FAULT 11 E(NO/YR)	0.3892E-05	0.2155E-05	0.1224E-05	0.7105E-06	0.4212E-06
FAULT 12 E(NO/YR)	0.4802E-06	0.2361E-06	0.1201E-06	0.6297E-07	0.3395E-07
FAULT 13 E(NO/YR)	0.5842E-06	0.2910E-06	0.1499E-06	0.7954E-07	0.4336E-07
FAULT 14 E(NO/YR)	0.2765E-06	0.1410E-06	0.7421E-07	0.4019E-07	0.2234E-07
FAULT 15 E(NO/YR)	0.1023E-07	0.4849E-08	0.2385E-08	0.1212E-08	0.6346E-09
FAULT 16 E(NO/YR)	0.1193E-07	0.5398E-08	0.2542E-08	0.1241E-08	0.6254E-09

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   FAULT  17 E(NO/YR)      0.2014E-070.9328E-080.4489E-080.2236E-080.1149E-08
   FAULT  18 E(NO/YR)      0.6692E-080.3055E-080.1452E-080.7146E-090.3631E-09
   FAULT  19 E(NO/YR)      0.1526E-080.6738E-090.3105E-090.1486E-090.7346E-10
   FAULT  20 E(NO/YR)      0.7109E-060.3775E-060.2062E-060.1156E-060.6634E-07
   FAULT  21 E(NO/YR)      0.8604E-090.3697E-090.1660E-090.7749E-100.3739E-10
   TOTAL     E(NO/YR)      0.1590E-020.1129E-020.8091E-030.5846E-030.4258E-03
   TOTAL RISK              0.1589E-020.1129E-020.8088E-030.5844E-030.4257E-03

     SPECIFIED RISKS:         0.010000  0.005000  0.002105  0.001000  0.000404
     ESTIMATED LN AMP.  :      -0.503    -0.246     0.019     0.211 *********
      ESTIMATED AMP. (g):     0.60448   0.78217   1.01906   1.23539 *********

 BOORE ET AL(1997) NEHRP D (250)2
      AMPLITUDES (g):
0.1000E+000.2000E+000.3000E+000.4000E+000.5000E+000.6000E+000.7000E+000.8000E+000.9000E+000.1000E+01
      LN (AMPLITUDE):       -2.30     -1.61     -1.20     -0.92     -0.69     -0.51     -0.36     -0.22
-0.11      0.00
   FAULT   1 E(NO/YR)
0.1738E-020.1123E-020.6868E-030.4049E-030.2359E-030.1380E-030.8161E-040.4899E-040.2989E-040.1854E-04
   FAULT   2 E(NO/YR)
0.4338E-020.2605E-020.1647E-020.1019E-020.6249E-030.3842E-030.2383E-030.1497E-030.9526E-040.6148E-04
   FAULT   3 E(NO/YR)
0.2454E-020.1417E-020.8425E-030.4866E-030.2787E-030.1606E-030.9378E-040.5566E-040.3361E-040.2065E-04
   FAULT   4 E(NO/YR)
0.7471E-020.3306E-020.1544E-020.7469E-030.3748E-030.1949E-030.1048E-030.5810E-040.3311E-040.1935E-04
   FAULT   5 E(NO/YR)
0.2384E-010.1538E-010.9810E-020.6045E-020.3670E-020.2227E-020.1362E-020.8429E-030.5288E-030.3364E-03
   FAULT   6 E(NO/YR)
0.1337E-010.5016E-020.1918E-020.7723E-030.3300E-030.1491E-030.7081E-040.3513E-040.1812E-040.9671E-05
   FAULT   7 E(NO/YR)
0.3100E-020.1082E-020.3656E-030.1297E-030.4917E-040.1990E-040.8541E-050.3862E-050.1829E-050.9016E-06
   FAULT   8 E(NO/YR)
0.1606E-020.4132E-030.1111E-030.3322E-040.1100E-040.3979E-050.1553E-050.6464E-060.2844E-060.1313E-06
   FAULT   9 E(NO/YR)
0.6755E-030.1211E-030.2480E-040.6002E-050.1675E-050.5248E-060.1809E-060.6753E-070.2696E-070.1140E-07
   FAULT  10 E(NO/YR)
0.1407E-020.2469E-030.5098E-040.1250E-040.3532E-050.1120E-050.3906E-060.1473E-060.5937E-070.2533E-07
   FAULT  11 E(NO/YR)
0.7694E-020.3460E-020.1511E-020.6601E-030.2967E-030.1385E-030.6717E-040.3380E-040.1759E-040.9442E-05
   FAULT  12 E(NO/YR)
0.1022E-010.1920E-020.4115E-030.1032E-030.2960E-040.9488E-050.3334E-050.1265E-050.5121E-060.2193E-06
   FAULT  13 E(NO/YR)
0.1116E-010.2221E-020.5100E-030.1358E-030.4104E-040.1376E-040.5028E-050.1975E-050.8255E-060.3638E-06
   FAULT  14 E(NO/YR)
0.2973E-020.5964E-030.1362E-030.3613E-040.1090E-040.3654E-050.1337E-050.5263E-060.2204E-060.9740E-07
   FAULT  15 E(NO/YR)
0.2870E-030.3726E-040.5980E-050.1194E-050.2844E-060.7788E-070.2388E-070.8035E-080.2923E-080.1136E-08
   FAULT  16 E(NO/YR)
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0.6740E-030.5163E-040.5846E-050.9027E-060.1754E-060.4059E-070.1078E-070.3200E-080.1041E-080.3663E-09
FAULT 17 E(NO/YR)
0.8541E-030.8899E-040.1232E-040.2207E-050.4821E-060.1229E-060.3546E-070.1131E-070.3925E-080.1462E-08
FAULT 18 E(NO/YR)
0.5311E-030.5530E-040.7768E-050.1414E-050.3138E-060.8121E-070.2375E-070.7677E-080.2696E-080.1016E-08
FAULT 19 E(NO/YR)
0.2739E-030.1921E-040.2148E-050.3341E-060.6583E-070.1549E-070.4184E-080.1263E-080.4179E-090.1492E-09
FAULT 20 E(NO/YR)
0.3366E-020.1017E-020.3126E-030.1036E-030.3727E-040.1446E-040.5991E-050.2628E-050.1212E-050.5834E-06
FAULT 21 E(NO/YR)
0.3218E-030.2317E-040.2596E-050.4019E-060.7865E-070.1836E-070.4923E-080.1476E-080.4847E-090.1719E-09
TOTAL E(NO/YR)
0.9835E-010.4020E-010.1992E-010.1070E-010.5996E-020.3459E-020.2045E-020.1235E-020.7613E-030.4779E-03
TOTAL RISK
0.9367E-010.3940E-010.1972E-010.1064E-010.5978E-020.3453E-020.2043E-020.1235E-020.7610E-030.4778E-03

AMPLITUDES (g):	0.1100E+01	0.1200E+01	0.1300E+01	0.1400E+01	0.1500E+01
LN (AMPLITUDE):	0.10	0.18	0.26	0.34	0.41
FAULT 1 E(NO/YR)	0.1168E-04	0.7474E-05	0.4852E-05	0.3194E-05	0.2129E-05
FAULT 2 E(NO/YR)	0.4022E-04	0.2666E-04	0.1789E-04	0.1216E-04	0.8351E-05
FAULT 3 E(NO/YR)	0.1290E-04	0.8191E-05	0.5279E-05	0.3450E-05	0.2286E-05
FAULT 4 E(NO/YR)	0.1157E-04	0.7066E-05	0.4398E-05	0.2786E-05	0.1793E-05
FAULT 5 E(NO/YR)	0.2171E-03	0.1420E-03	0.9409E-04	0.6314E-04	0.4287E-04
FAULT 6 E(NO/YR)	0.5322E-05	0.3010E-05	0.1745E-05	0.1034E-05	0.6256E-06
FAULT 7 E(NO/YR)	0.4609E-06	0.2433E-06	0.1322E-06	0.7373E-07	0.4211E-07
FAULT 8 E(NO/YR)	0.6326E-07	0.3163E-07	0.1635E-07	0.8704E-08	0.4761E-08
FAULT 9 E(NO/YR)	0.5071E-08	0.2356E-08	0.1138E-08	0.5687E-09	0.2932E-09
FAULT 10 E(NO/YR)	0.1135E-07	0.5314E-08	0.2584E-08	0.1300E-08	0.6744E-09
FAULT 11 E(NO/YR)	0.5212E-05	0.2952E-05	0.1711E-05	0.1014E-05	0.6121E-06
FAULT 12 E(NO/YR)	0.9860E-07	0.4627E-07	0.2254E-07	0.1136E-07	0.5903E-08
FAULT 13 E(NO/YR)	0.1679E-06	0.8071E-07	0.4022E-07	0.2070E-07	0.1097E-07
FAULT 14 E(NO/YR)	0.4509E-07	0.2174E-07	0.1087E-07	0.5611E-08	0.2982E-08
FAULT 15 E(NO/YR)	0.4674E-09	0.2022E-09	0.9135E-10	0.4292E-10	0.2087E-10
FAULT 16 E(NO/YR)	0.1376E-09	0.5436E-10	0.2237E-10	0.9563E-11	0.4092E-11
FAULT 17 E(NO/YR)	0.5789E-09	0.2417E-09	0.1057E-09	0.4778E-10	0.2243E-10
FAULT 18 E(NO/YR)	0.4064E-09	0.1713E-09	0.7554E-10	0.3462E-10	0.1635E-10
FAULT 19 E(NO/YR)	0.5680E-10	0.2279E-10	0.9525E-11	0.4094E-11	0.1780E-11
FAULT 20 E(NO/YR)	0.2920E-06	0.1512E-06	0.8071E-07	0.4429E-07	0.2492E-07
FAULT 21 E(NO/YR)	0.6505E-10	0.2595E-10	0.1077E-10	0.4619E-11	0.1953E-11
TOTAL E(NO/YR)	0.3051E-03	0.1979E-03	0.1303E-03	0.8694E-04	0.5876E-04
TOTAL RISK	0.3051E-03	0.1979E-03	0.1303E-03	0.8694E-04	0.5876E-04

SPECIFIED RISKS:	0.010000	0.005000	0.002105	0.001000	0.000404
ESTIMATED LN AMP. :	-0.892	-0.634	-0.365	-0.172	0.036
ESTIMATED AMP. (g):	0.40978	0.53058	0.69387	0.84212	1.03628

CLOSEST DISTANCES BETWEEN SITE AND FAULT RUPTURES

NO.	FAULT NAME	CD_1DRP	CD_2DRP	CDIST	CLODIS	CD_EPI	CD_HYPO	
1	HOLSER	2.4	0.0	2.2	2.2	0.1	2.2	km
2	NORTHRIDGE (E. Oak Ridge)	4.6	4.6	6.8	6.8	5.4	7.8	km
3	SAN GABRIEL	5.9	5.9	5.9	5.9	5.9	6.1	km
4	OAK RIDGE (Onshore)	8.3	8.3	8.3	8.3	9.4	9.6	km
5	SANTA SUSANA	9.6	0.4	7.9	7.9	1.1	7.9	km
6	SAN CAYETANO	11.4	11.4	11.4	11.4	12.6	12.6	km
7	SIERRA MADRE (San Fernando)	19.9	14.8	17.5	17.5	15.9	18.5	km
8	SIMI-SANTA ROSA	20.9	18.9	20.6	20.6	20.0	21.6	km
9	VERDUGO	27.0	26.8	26.9	26.9	27.9	28.0	km
10	SANTA YNEZ (East)	30.4	28.6	30.1	30.1	29.4	30.8	km
11	SAN ANDREAS - 1857 Rupture	33.0	33.0	33.0	33.0	33.0	33.2	km
12	SAN ANDREAS - Mojave	33.0	33.0	33.0	33.0	33.0	33.0	km
13	SAN ANDREAS - Carrizo	33.0	33.0	33.0	33.0	33.0	33.0	km
14	SIERRA MADRE	35.5	31.6	33.5	33.5	32.8	34.6	km
15	MALIBU COAST	42.6	39.2	41.1	41.1	39.5	41.1	km
16	HOLLYWOOD	43.5	38.7	40.9	40.9	39.1	41.0	km
17	SANTA MONICA	43.6	40.2	42.1	42.1	40.5	42.1	km
18	VENTURA - PITAS POINT	46.9	46.9	46.9	46.9	48.0	48.0	km
19	M.RIDGE-ARROYO PARIDA-SANTA ANA	48.6	48.6	48.6	48.6	49.8	49.8	km
20	ANACAPA-DUME	49.1	32.9	38.4	38.4	34.2	39.2	km
21	NEWPORT-INGLEWOOD (L.A.Basin)	49.8	49.8	49.8	49.8	50.9	50.9	km

EXPLANATION

CD_1DRP = Closest distance to projection of rupture area along fault trace.
CD_2DRP = Closest distance to surface projection of the rupture area.
CDIST = Closest distance to seismogenic rupture.
CLODIS = Closest distance to subsurface rupture.
CD_EPI = Closest epicentral distance.
CD_HYPO = Closest hypocentral distance.

APPENDIX C

Attenuation Relationship	Unweighted Peak Acceleration (6.5M)			Weighted Peak Acceleration (7.5M)		
	475 Yr Return Period (g)	2475 Yr Return Period		475 Yr Return Period (g)	2475 Yr Return Period	
		Raw Data (g)	Two-Thirds (g)		Raw Data (g)	Two-Thirds (g)
Boore et al (1997)	1.01906	1.51646	1.01096	0.69387	1.03628	0.69085
Sadigh et al (1997)	0.78175	1.13498	0.75665	0.52372	0.75463	0.50308
Campbell & Bozorgnia	0.80553	1.10252	0.73501	0.54633	0.74883	0.49922
Average	**0.87**	1.25132	**0.83**	**0.59**	0.84658	**0.56**

Appendix D

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

March 28, 2008

Newhall Land and Farming Company
23823 West Valencia Boulevard
Valencia, California 91355

Job No. 99-802-21

Attention: Mr. Fred MacMurdo

Subject: Geotechnical Grading Plan Review
Proposed Offsite Zone 1A Water Tank Site
Vesting Tentative Tract Map No. 53108
Los Angeles County, California

References: See Attached List

Gentlemen:

This report presents the results of our geotechnical grading plan review conducted for the Zone 1A Water Tank Site (herein referred to as the tank site), Vesting Tentative Tract Map No. 53108, located in Los Angeles County, California. The scope of this study was planned in consultation with Mr. Fred MacMurdo of Newhall Land and Farming Company (NLF) as outlined in our proposal dated March 12, 2008 (Proposal No. P025-2008-21). We are in receipt of the *Proposed Offsite Zone 1A Water Tank Grading Exhibit* prepared by Psomas. This plan, prepared at a scale of 1 inch equals 100 feet (1"=100'), is designated as Sheet 6 of 13 of the "Major Land Division, Vesting Tentative Tract Map No. 53108" plan. The geotechnical data collected as part of our investigation is presented on the Psomas plan and included in this report as the Geotechnical Map, Figure 1.

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SCOPE OF WORK

The scope of work for the geotechnical grading plan review was described in our March 12, 2008 proposal to NLF. The rough grading plan review consisted of the following:

- a review of data, aerial photographs, geologic literature, and previous geotechnical and engineering geologic reports;
- engineering geologic evaluation of the site to asses the impact of geologic conditions on future site development;
- engineering evaluation of the geotechnical data to develop recommendations for mass grading earthwork; and
- drafting and report preparation.

PROPOSED CONSTRUCTION

The proposed tank site is located along an easterly descending ridge approximately ¼ mile north of Franklin Parkway, west of the U.S. Postal Service Distribution Center. Based on plans provided our office by Psomas, the proposed tank site pad will be graded to an approximate elevation of 1,172 feet above mean sea level (msl). Based on the proposed grading, a 2:1 (horizontal:vertical) cut slope on the order of 110 feet high will ascend from the north and west sides of the tank pad. A 20± feet high 2:1 combination cut and fill slope will descend from the south side of the tank pad. The proposed pad will support a water tank approximately 120 feet in diameter. The tank access road will extend along the south side of the tank and will include a 10±feet high, 2:1 graded slope along the north side of the road.

SUBSURFACE EXPLORATIONS

Subsurface data was available for the site and adjacent areas from a prior R. T. Frankian & Associates (RTF&A) investigation (RTF&A, 2001b). The logs of the RTF&A borings and test pits pertinent to the tank site are presented in Appendix A.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from prior borings drilled on the site. The prior laboratory data has been presented in our prior investigation (RTF&A, 2001b) and is included in this report as Appendix B.

GEOLOGY

REGIONAL GEOLOGY

The tank site is located at the eastern end of the Ventura basin within the Transverse Ranges geomorphic province of California. The Ventura basin consists of a narrow, elongate sedimentary trough extending from the Santa Barbara Channel on the west to the San Gabriel fault on the east. The axis of the trough trends east-west, reflecting the overall east-west trend of the Transverse Ranges, and generally coincides with the Santa Clara River Valley. The Ventura basin has been an area of subsidence and sediment accumulation since the beginning of the Tertiary period, with the present trough-like form developing near the beginning of the Miocene epoch (Winterer and Durham, 1962).

The structure of the basin is defined as a highly folded "synclinorium" formed by north-south compressional forces (Kew, 1924), and containing a maximum 50,000± feet of marine and nonmarine Tertiary through Quaternary age sedimentary

rocks (Bailey and Jahns, 1954). Within the Santa Clarita Valley the primary sedimentary rock formations are the Pico and Saugus Formations. The Pico Formation outcrops along the northern flanks of the Santa Susana Mountains and in the Hasley Canyon-Val Verde area. The Saugus Formation overlies the Pico Formation and comprises most of the hills of the valley between Newhall and Castaic. Other geologic materials exposed within the valley include Pleistocene fanglomerate and terrace deposits (Oakeshott, 1958; Winterer and Durham, 1962), exposed in the southern and southwestern portion of the valley, and Holocene alluvium mantling the valley floor.

The Pico and Saugus Formations have been deformed into a series of closely spaced anticlines and synclines whose moderately to steeply dipping flanks are broken by the Holser fault and cut off diagonally by the San Gabriel fault (Bailey and Jahns, 1954). The San Gabriel fault, the dominant geologic feature in the Santa Clarita Valley, forms the eastern Ventura basin boundary, and separates the Ventura basin from the structurally similar Soledad basin.

SITE GEOLOGY

General: The tank site is situated on the northerly limb of the Ventura basin "synclinorium", approximately 0.6 mile southwest of the Holser fault. Bedrock beneath the site, and in the surrounding area, has been deformed along the Holser fault, as indicated by several tight folds that trend subparallel to the fault trace. The east-west trend of the Holser fault, and the subparallel fold axes reflect the north-south compressional tectonic forces that have shaped the region.

The seismically active San Gabriel fault zone, trending northwest-southeast through the Santa Clarita Valley, is located approximately 3 miles northeast of the tank site.

Geologic Materials: Geologic materials observed within the tank site include the Saugus Formation, terrace deposits, landslide deposits, and man-made deposits. The areal extent of the various geologic units are depicted on the Geotechnical Map, (Figure 1) and are described below.

Saugus Formation (TQs): The Plio-Pleistocene age Saugus Formation (map unit "TQs") underlies the tank site. As observed in outcrops and exploratory borings and test pits, the Saugus Formation is composed of poorly sorted, weakly cemented to moderately well cemented sandstone and pebbly sandstone, with alternating beds of poorly to moderately indurated siltstone, sandy siltstone/silty sandstone, and claystone. Sandstone, pebbly sandstone, and siltstone constitute the dominant rock types. Sandstones are primarily light brown to light gray/yellowish gray fine to coarse grained, with some thin interbedded very fine to fine grained laminated and cross-bedded sandstone. The siltstone and claystone are generally reddish-brown.

Terrace Deposits (Qt): Remnant stream-terrace deposits (map unit "Qt") are situated along the west side of the existing water tank access road and east of the proposed tank site. The Pleistocene age deposits are composed of yellowish brown to brown, poorly bedded friable sand, gravel and silt.

Landslides (Qls): Numerous landslides (map unit "Qls") were identified within the area of the tank site, with two landslides occurring within the area of the proposed tank site and access road grading. The landslides are generally composed of disturbed bedrock materials derived from the underlying Saugus Formation.

As well as the bedrock landslides, a surficial failure was identified northeast of the tank site. The surficial failure is probably limited to five to eight feet in depth and incorporates weathered, less competent near-surface earth materials.

Man-made Deposits (af and cef): Man-made deposits include artificial fill and certified engineered fill (map units "af" and "cef", respectively) placed during construction of the existing tank site and the Post Office. The artificial fill is located in the existing debris basins along the north side of the existing tank access road, and bordering the Post Office site. The certified engineered fill is located beneath the existing water tank, and along potions of the tank access road.

Geologic Structure: Saugus Formation units beneath the tank site have been folded into an east-west trending, easterly-plunging syncline. Sedimentary beds forming the north limb of the synclinal fold strike east-northeast and dip 37 to 60 degrees southerly towards the fold axis. Beds on the south limb strike west-northwest to northwest, dipping northerly 24 to 37 degrees towards the axis.

The approximate location of the axial trace of the syncline is shown on Figure 1. A stereonet analyses of the structural configuration of the syncline indicates that the syncline plunges approximately 12 to 15 degrees to the east.

SLOPE STABILITY

GENERAL

Grading for the proposed tank site will include a cut pad at an approximate elevation of 1,172 feet, which will create an ascending cut slope on the north and west side of the tank pad (designated Cut Slope CS-1), and a descending cut slope

from the south side (Cut Slope CS-2). A 2:1 combination fill and cut slope (CS-3) will be graded along the north side of the tank site access road.

The maximum cut slope height is approximately 110 feet. Fill slopes are planned at gradients of 2:1. The maximum proposed fill slope height is approximately 20 feet.

SHEAR STRENGTH PARAMETERS

Direct shear tests were performed for the previous geotechnical investigation of the tank site (RTF&A, 2001a and 2001b). In addition, we reviewed shear strength parameters for other jobs we have done in the vicinity of the site. Presented below are the recommended shear strengths for use at the subject site.

SUMMARY OF SHEAR STRENGTH PARAMETERS

MATERIAL	Static Cohesion (psf)	Static ϕ (degrees)
Landslide Failure Plane Material	200	16
Clayey Bedding Plane Material	250	20
Clayey Bedding Plane Material (Seismic)	375	19
Saugus Cross Bedding	640	32
Alluvium	100	40
Compacted Fill	300	32

CUT SLOPE CS-1

Cut Slope CS-1 will be graded as a 110 feet high, east- to south-facing 2:1 cut slope. This slope will be underlain by sedimentary rock units of the Saugus Formation. The Saugus Formation can range from massive to thinly bedded sedimentary rock units of sandstone, conglomerate, siltstone, and

claystone/mudstone. Bedding planes within the Saugus Formation are poorly to moderately-well developed, and can constitute planes of weakness, particularly where sandstone/conglomerate beds are in contact with claystone. Where bedding is adversely oriented, or "daylighted," with respect to natural or cut slopes, potential for bedding plane, or "block-glide," failure exists.

The subsurface geologic conditions relative to CS-1 are depicted on Geologic Sections A-A' and B-B' (Figures 2.1 and 2.2, respectively). Geologic Section A-A' illustrates geologic conditions parallel to the axis of the easterly-plunging syncline, for the segment of the cut slope facing to the east. Bedding in this area strikes northeast and dips 40 degrees to 45 degrees to the southeast. The apparent dip, with respect to the east-facing cut slope segment, is 32 degrees towards the east. As illustrated, the proposed east-facing segment of the cut slope is considered to be grossly stable from a geologic standpoint, as the angle of apparent dip (32 degrees) is steeper than the proposed 2:1 (or approximately 26 degrees) cut-slope.

Geologic Section B-B' depicts the geologic conditions for CS-1 perpendicular to the synclinal axis, and across the south-facing portion of the cut-slope. Bedding with respect to the south-facing segment of CS-1 exhibits an apparent dip of 37 degrees to the south. This bedding orientation dips steeper than the 2:1 (26 degrees) cut slope gradient and the south-facing segment of CS-1 is considered grossly stable from a geologic standpoint.

CUT SLOPE CS-2

Cut Slope CS-2 is planned as a 20 feet high, south-facing combination cut and fill slope that will descend from the tank pad to the tank access road. Cut slope CS-2

will expose landslide debris and, at depth, Saugus Formation units. The landslide debris is loose and inherently unstable. As depicted on Geologic Section B-B', a portion of CS-2, and the south edge of the proposed cut pad, will expose landslide deposits which have failed in the trough of the syncline.

The landslide debris should be completely removed and the entire slope constructed as a 2:1 stability fill slope which will mitigate the existing landslide. The estimated landslide depth, based on Test Pit TP-7, is 5 to 10 feet. The landslide removal will likely extend into a portion of the tank pad. Accordingly, construction of the fill slope should include fill placement to restore the pad grade. The keyway for the proposed 2:1 stability fill should measure a minimum of 15 feet wide, and be founded at least 3 feet into competent bedrock. Backdrains should be constructed at the fill/bedrock interface as addressed in the "CONLUSIONS AND RECOMMENDATIONS" section of the report and as shown on Figure 4 – Stability Fill Details for Grossly Stable Slopes.

CUT SLOPE CS-3

Cut Slope CS-3 will consist of a 10 feet high variable 2:1 to 3:1 combination fill and cut slope. The slope will likely encounter Saugus Formation units, landslide debris, and terrace deposits. The terrace deposits primarily consist of weakly to moderately cemented, massive to thickly bedded sand, silty sand, and gravel. Where noted, bedding in the terrace deposits is generally flat-lying. The terrace deposits are very granular and tend to lack cementation. Accordingly, these materials may be subject to erosion from water.

Geologic Section C-C' (Figure 2.3) depicts the proposed cut slope relative to the landslide. The landslide is potentially unstable, and the terrace deposits are susceptible to erosion. Therefore, it is recommended that the landslide be removed and the entire slope constructed as a stability fill slope which will mitigate the existing landslide. The estimated landslide depth, based on Test Pit TP-5, is 15 to 20 feet. The keyway for stability fill should measure a minimum of 15 feet wide, and be founded at least 3 feet into competent bedrock or terrace deposits. Backdrains should be constructed at the fill/bedrock interface. The stability fill, and backdrains, should be constructed in accordance with the recommendations presented in this report and as shown on Figure 4 – Stability Fill Details for Grossly Stable Slopes.

CONCLUSIONS AND RECOMMENDATIONS

GEOLOGIC CONSIDERATIONS

General: Based on the geologic data developed during the geotechnical investigation pertinent to the Zone 1A Water Tank Site, it is our opinion that the site may be developed as planned provided our recommendations are incorporated in the design of the project.

Faulting: No mapped active or potentially active faults underlie the proposed tank site, and the site is not within an Alquist-Priolo Earthquake Fault Zone, as established by California Division of Mines and Geology (CDMG) (1999). The closest active fault to the site is the San Gabriel fault, located approximately 3 miles northwest of the tank site.

Landslides: Two small landslides are located along the southern edge of the tank site and a portion of the tank access road. Complete removal of these landslides

will be required during grading operations. The estimated depth of these landslides ranges from 10 to 20 feet.

Liquefaction: The proposed tank pad is not within a liquefaction-mapped zone and by inspection of the materials encountered in the boring is not susceptible to liquefaction.

Erosion Protection: Friable sandstone beds are common within the Saugus Formation and have been identified at the site. If exposed in graded slopes, these beds could be subject to erosion and rilling, due to the lack of cementation. Under most circumstances, the erosion can be controlled by the establishment of vegetative cover over the slope upon completion of grading. Extensive or thick deposits of the friable beds may warrant construction of stability fills. The abundance of erosion susceptible beds should be determined during grading.

In order to reduce the potential for erosion, all graded slopes should be seeded or planted with proper ground cover as soon as possible, following grading. The ground cover should consist of drought-resistant, deep-rooting vegetation. A landscaping expert should be consulted for ground cover recommendations. If the potential for some erosion of the slopes is acceptable, at a minimum, a 3-foot deep, 5-foot wide ditch may be excavated at the toe of all slopes so when the materials do erode down the slope they do not threaten the tank or any other adjoining structures.

Drainage from the proposed slopes should be directed to non-erosive drainage devices. Surface drainage for the tank pad should be controlled and directed to drainage devices to minimize erosion of the slopes that descend from the proposed tank pad.

Rippability: Grading operations can be performed using conventional grading equipment. Heavy ripping may be needed when excavating well-cemented sandstone or conglomerate beds.

RESTRICTED USE AREAS

Based upon the geologic conditions and proposed grading, there are no Restricted Use Areas (RUAs) at the subject site.

GRADING

Site Preparation: Prior to performing earthwork, the existing vegetation and any deleterious debris should be removed from the site. Existing utility lines should be relocated or properly protected in-place. All unsuitable soils, landslide material, and uncertified fills in the areas of grading receiving new fill should be removed to competent earth materials and replaced with engineered fill.

Material for Fill: The on-site soils, less any debris or organic matter, may be used in required fills. Any expansive clays discovered should be mixed with non-expansive soils to result in a mixture having an expansion index less than 30 if they are to be placed within the upper 8 feet beneath proposed rough grades.

Rocks or hard fragments larger than 8 inches may not be placed in the fill without special treatment. Rocks or hard fragments larger than 4 inches shall not compose more than 25% of the fill or a lift. Soils containing more than 25% rock or hard fragments larger than 4 inches must be compacted with successive passes (e.g., with a sheepsfoot roller) until rock or hard fragments larger than 4 inches constitute less than 25% of the fill or lift.

Import material should consist of relatively non-expansive soils with an expansion index less than 30. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. The import material should be free of organic materials, debris, and cobbles larger than 8 inches. A bulk sample of potential import material, weighing at least 25 pounds, should be submitted to the Geotechnical Consultant of Record at least 48 hours in advance of fill operations. All proposed import materials should be approved by the Geotechnical Consultant of Record prior to being placed at the site.

Compaction: After the site is cleared and excavated as recommended, the exposed soils should be carefully observed for the removal of all unsuitable deposits. Next, the exposed soils should be scarified to a depth of 6 inches, brought to about 2% above optimum moisture content, and rolled with heavy compaction equipment. The upper 6 inches of exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM D 1557-02 Method of Compaction.

After compacting the exposed soils, all required fills should be placed in loose lifts, not more than 8 inches in thickness, and compacted to at least 90%. The moisture content of the fill soils at the time of compaction should be about 2 to 4% above optimum moisture content. Compacted fill should not be allowed to dry out before subsequent lifts are placed.

Rough exterior grades should be sloped so as not to direct water flow over slope faces. Finished exterior grades should be sloped to drain away from building foundations to prevent ponding of water.

Backfill: All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and any overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM D 1557-02 Method of Compaction.

Proposed Tank Pad Subgrade: It is anticipated that Saugus Formation bedrock materials exposed at pad grade may contain expansive claystone beds that could cause differential expansion. An 8-foot deep fill cap extending at least 8 feet outside of the footprint of the proposed water tank is recommended to reduce the effects of expansive soil conditions and the presence of the transition zone, associated with the recommended stability fill along the south side of the tank pad (see Figure 3). It is also recommended that the bedrock be removed and recompacted to a depth of at least 3 feet below proposed soil subgrade in exposed Saugus Formation areas to be paved. The soils generated by these over-excavations should be mixed with non-expansive soils to yield a relatively non-expansive mixture. Should the resulting fill soil still be expansive, special construction techniques and procedures may be required to reduce the potential for expansive soil related distress.

Tank Subdrain System: A subdrain system should be installed under the proposed water tank. A synthetic geomembrane (such as high-density polyethylene-HDPE) with a coefficient of permeability of 10^{-6} cm/sec or less be placed beneath the proposed tank site. A leakage collection and removal system should be provided between the tank bottom and the geomembrane. The tank manufacturer may also have other similar or equivalent drainage systems, which could be used in lieu of the

system recommended above. We should review the tank foundation and drainage plans after completion of rough grading operations.

Subdrains: A canyon subdrain is recommended to intercept and remove groundwater within the proposed canyon fill near the west end of the access road. The subdrain should extend up-canyon, with the drain inlet carried to within 15 feet of final pad grade. Specific subdrain location and length should be determined in the field during grading operations.

The subdrain should be surveyed by the Project Surveyor to establish line and grade during construction, and for future location reference. Subdrain excavation and placement should be observed by our geologist.

The subdrains should be installed in accordance with the manufacturer's specifications. A minimum 2% gradient is to be maintained in the subdrain pipes. There should be at least 8 uniformly spaced sets of 2 perforations per lineal foot of pipe. The width of the perforations should not exceed 1/16 of an inch. If PVC pipe with drilled perforations is utilized, the diameter of the holes should not exceed 3/8 of an inch, if gravel and filter fabric is used or 1/8 inch diameter if Los Angeles County Flood Control District (LACFCD) Designation F-1 Filter Material is used. When constructing the subdrain, the pipe should be placed so that the drilled perforations are positioned on the bottom half of the pipe. The upstream end of subdrains should be capped. The final 20 feet of pipe at the downstream end of canyon, stabilization, buttress, and side hill fills shall not be slotted or perforated. Provisions should be made at all times during construction to prevent damage to the subdrain from construction equipment, and to prevent soils from being washed into an exposed subdrain by surface waters.

For runs up to 500 feet, subdrains for the bottom of canyon fills should consist of at least 6-inch diameter pipe. For runs of 500 to 1,500 feet, 8-inch diameter pipe shall be used. For runs over 1,500 feet, 10-inch diameter pipe shall be used.

Canyon subdrains should be installed in a rectangular trench excavated to expose competent material and shall be approved by the Geotechnical Consultant of Record. The slotted subdrains should be surrounded by at least 3 cubic feet per lineal foot of granular filter material. The granular filter material for subdrains should meet the LACFCD Designation F1, or have a gradation approved by the Geotechnical Consultant of Record prior to placement. There should be at least 6 inches of compacted granular filter material on all sides of the pipe (See Figure 5). As an alternative, drains may be placed in a "V" ditch only after approval by the Engineering Geologist (see Figure 6).

As an alternative to the granular filter material, ¾ inch diameter gravel may be placed around the pipe. The gravel should be separated from the surrounding soils by a filter fabric such as Mirafi 140N, or equivalent, wrapped around the gravel ("burrito wrapped").

Stability Fill Backdrains: Backdrains should be installed at the backcut of the stability fills. The backdrains should consist of 4-inch diameter, or larger, perforated pipe surrounded by filter material. Non-perforated drain outlets should be provided at vertical intervals not exceeding 15 feet and horizontal intervals not exceeding 100 feet. The exact location of subdrains should be determined in the field by the Geotechnical Consultant of Record after the backcut has been made.

Geotechnical Observation: The grading operations should be observed by the Geotechnical Consultant of Record. The Geotechnical Consultant of Record's representative should have at least the following duties:

- observe the excavation so that any necessary modifications based on variations in the soil/rock conditions encountered can be made;

- observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proof-rolling and delineation of areas requiring overexcavation;

- evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary;

- observe the fill and backfill for uniformity during placement;

- test fill for field density and compaction to determine the percentage of compaction achieved during fill placement; and,

- geologic observation of all cut slopes, keyways, backcuts and geologic exposures during grading to ascertain that conditions conform to those anticipated in the report.

Once the necessary grading permits are obtained, the governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the regulatory agencies.

FOUNDATIONS

General: The proposed water tank and footings for auxiliary structures or retaining walls may be supported on continuous or individual spread footings established in properly compacted fill. It is recommended that a formal review of foundation plans and foundation loading details be performed by the Geotechnical Consultant of Record when plans become available to verify the applicability of the recommendations contained herein. The recommendations presented below should be considered preliminary and should be finalized after review of foundation plans. The preliminary design values are based upon this investigation, our experience with the soils in the area, and with the site preparation and grading recommendations for this project.

Bearing Capacity: It is assumed that the proposed water tank will be founded at approximately final planned grades shown on the Geotechnical Map. Individual or continuous footings should have a width of at least 12 inches and be placed at a depth of at least 24 inches below the lowest final adjacent grade.

Shallow footings established in certified compacted fill may be designed using a bearing value of 2,500 pounds per square foot (psf). The recommended bearing value is a net value, and the weight of concrete in the footings may be taken as 50 pounds per cubic foot (pcf). The weight of soil backfill may be neglected when determining the downward loads from the footings. A one-third increase in the bearing value may be used when considering wind or seismic loads.

While the actual bearing value of the fill placed at the site will depend on the materials used and the compaction methods employed, the quoted bearing value will

be applicable if acceptable soils are used and are compacted as recommended. The bearing value of the fill should be confirmed during grading.

Lateral Resistance: Lateral loads may be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 applied to the dead loads may be used between the footings, floor slabs and the supporting soils. The passive resistance of properly compacted fill soils may be assumed to be equal to the pressure developed by a fluid with a density of 250 pcf. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

Settlement: Provided that foundations are underlain by compacted fill soils as recommended, and maximum loads do not exceed 3.5 kips per lineal foot, we estimate that the maximum settlement will be about 1 inch (at the center) and differential settlement within a horizontal distance of 30 feet will be less than 0.75 inches.

Foundation Observations: To verify the presence of satisfactory soils at foundation design elevations, the excavations should be observed by the Geotechnical Consultant of Record. Excavations should be deepened as necessary to extend into satisfactory soils. Where the foundation excavations are deeper than four feet, the sides of the excavations should be sloped back at ¾:1 or shored for safety.

RETAINING WALLS

Lateral Earth Pressures: For design of cantilevered walls below grade, where the surface of the backfill is level and the retained height of soils is less than 15 feet,

it may be assumed that drained, nonexpansive soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pcf. Where the surface of the backfill is inclined at 2:1, it may be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 45 pcf. In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharges due to buildings, walls, and storage or traffic loads. A drainage system, such as weepholes or a perforated pipe should be provided behind the walls to prevent the development of hydrostatic pressure. Recommendations for wall drains are presented as follows.

If a drainage system is not installed, the walls should be designed to resist a hydrostatic pressure equal to that developed by a fluid with a density of 80 pcf against the full height of the wall.

In addition to the recommended earth and hydrostatic pressures, the upper 10 feet of walls adjacent to vehicular traffic areas should be designed to resist a uniform lateral pressure of 100 psf. This pressure is based on an assumed 300 psf surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected. Special recommendations will be required where expansive soils are to be retained.

Retaining Wall Drainage: A drainage system should be provided behind retaining walls, or the walls should be designed to resist hydrostatic pressures. The drainage system could consist of a 4-inch diameter perforated pipe placed 6 inches from the base of the wall, with the perforations down, and connected to an outlet device. The pipe should be sloped at least 1 inch per 50 feet and surrounded on all sides by at least 6 inches of clean gravel. The gravel should be "burrito-wrapped"

with filter fabric, such as Mirafi 140N, or equivalent. As an alternative to the gravel and filter fabric, filter material meeting the requirements of Los Angeles County Flood Control District Designated F-1 Filter Material, and slotted pipe, may be used. The backside of the wall should be waterproofed.

A vertical six-inch wide gravel chimney drain, or a drainage geocomposite such as Miradrain, should be placed against and behind retaining walls that are higher than 3 feet. The top of the back drain should be capped with 18 inches of on-site soils. Details regarding the drainage system, chimney drain, Miradrain, waterproofing, and soil cap as shown on the attached Retaining Wall Details, Figures 7.1 and 7.2.

The installed drainage system should be observed by the Geotechnical Consultant of Record prior to backfilling the system. Inspection of the drainage system may also be required by the reviewing governmental agencies.

The drainage system should be observed by the Geotechnical Consultant of Record prior to backfilling the system. Inspection of the drainage system may also be required by the reviewing governmental agencies.

PAVEMENT DESIGN

Following the completion of grading operations, samples of the on-site soil should be obtained from near final grade in pavement areas to perform R-value tests. The following preliminary pavement section recommendations are based on the assumption that the on-site soils have an R-value of 18. The final pavement section recommendations could vary depending on the results of the actual R-value test results obtained from samples obtained from the pavement areas.

TRAFFIC INDEX	ASPHALT THICKNESS (INCHES)	BASE COURSE (CAB) THICKNESS (INCHES)	BASE COURSE (CMB) THICKNESS (INCHES)
4	3	4	6
6	4	7	9
8	5	11	13
10	7	18	20
11	7	21	23

Base course material should consist of crushed aggregate base (CAB), as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction ("Greenbook"). If crushed miscellaneous base (CMB) is used, it should meet the specifications outlined in Section 200-2.4 of the "Greenbook". Base course should be compacted to at least 95% of the maximum dry density of that material.

Base course material should be purchased from a supplier who will certify that the base course will meet or exceed the specifications in the "Greenbook", as indicated. We could, at your request, perform sieve analysis and sand equivalency tests on material delivered to the site which appears suspect. Additional tests could be performed, upon request, to determine if the material is in compliance with the specifications.

The pavement section recommendations presented above are based upon assumed Traffic Index values. RTF&A does not take responsibility for the numerical determination of the Traffic Index values or the areas where they apply within the site. We would be pleased to provide pavement section recommendations for alternative Traffic Index values upon request.

GEOTECHNICAL CONSULTANT OF RECORD

This report has been prepared assuming that RTF&A will perform all geologic and geotechnically-related field observations and testing. If the recommendations presented in this report are to be utilized, but observation of the grading activities is performed by others, the parties performing the work must review this report and assume responsibility for recommendations contained herein or provide their own recommendations. That party would then assume the title "Geotechnical Consultant of Record" for the project.

A representative of the Geotechnical Consultant of Record should be present to observe all grading operations as well as test compacted fills. A report presenting the results of these observations and related testing should be issued upon completion of these operations.

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The following are attached and complete this report.

- References
- Geotechnical Map – Figure 1 (in pocket)
- Geotechnical Sections - Figures 2.1 through 2.3
- Transition Zone Detail - Figure 3
- Stability Fill Details for Grossly Stable Slopes – Figure 4
- Typical Canyon Subdrain - Figure 5
- Alternate Canyon Subdrain - Figure 6
- Retaining Wall Details – Figure 7.1 and 7.2
- Appendix - Field Explorations and Laboratory Testing
  - Log of Boring B-1
  - Test Pit Logs TP-1 through TP-9
  - Summary of Shear Test Data – (2 pages)

Respectfully submitted,

R. T. FRANKIAN & ASSOCIATES

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Principal Geotechnical Engineer

and: Timothy P. Latiolait
Principal Engineering Geologist

AWR/TPL/sjc

Distribution: (3) Newhall Land and Farming Company
Attn: Mr. Fred MacMurdo
(4) Psomas
Attn: Mr. Erik Winata
(with 3 wet-signed copies and 1 CD in PDF Format for submittal to Los Angeles County Department of Public Works)

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

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Geotechnical Map

PREPARED FOR
Newhall Land and Farming Company

Project Location
Zone 1A Water Tank Site Grading
Castaic, California

SCALE	DRAWN BY	CHECKED BY
AS SHOWN	MN	
DATE		JOB NO.
3-28-08	Figure 1	99-802-21

1329 SCOTT ROAD
BURBANK, CA. 91504
TEL: (818) 531-1501
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RTFA
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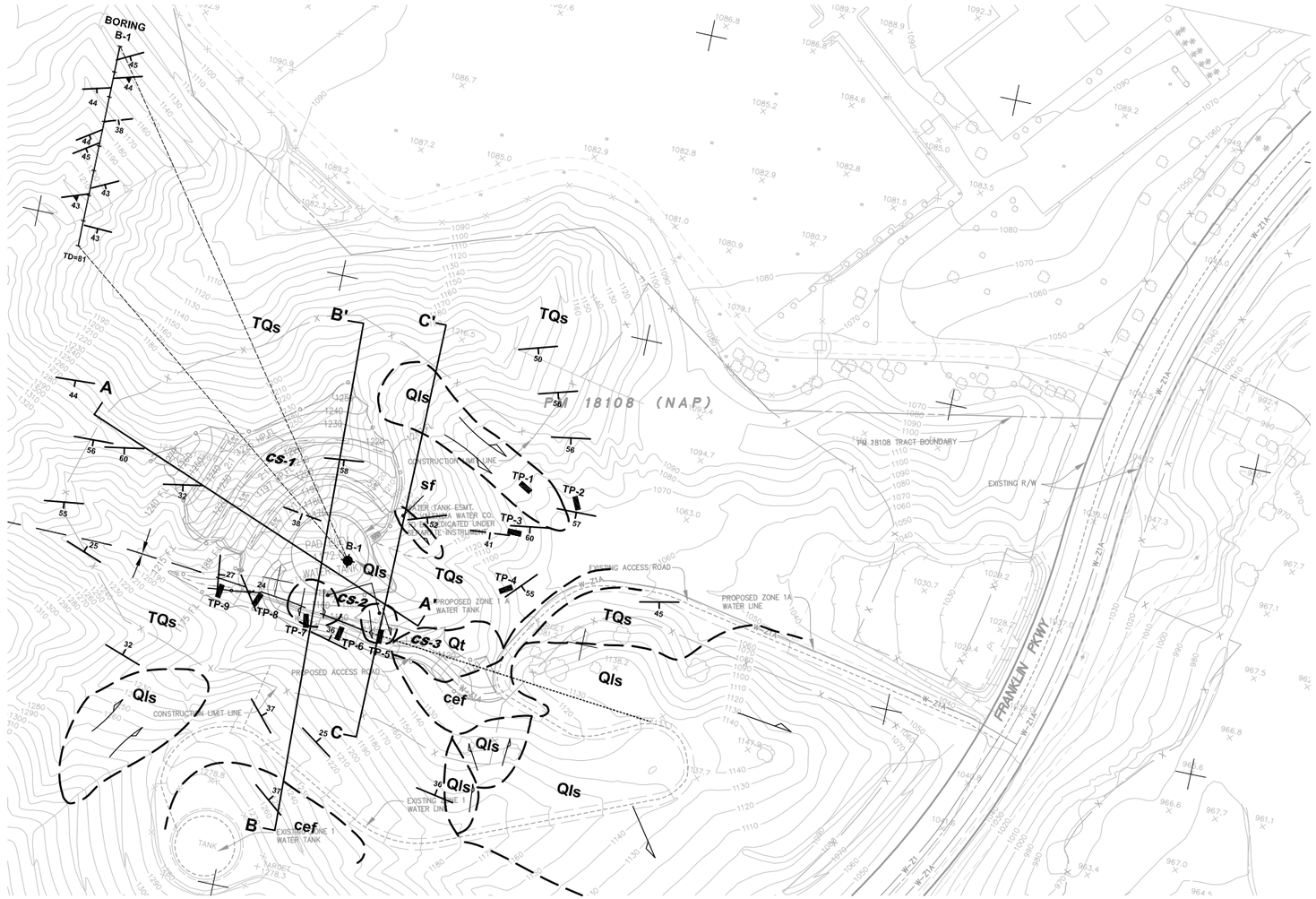


EXPLANATION

cef	Certified Engineered Fill
sf	Surficial Failure
Qls	Landslide Deposits
Qt	Terrace Deposits
TQs	Saugus Formation
	Geologic Contact, approximately located; queried where questionable
	Axial Trace of Syncline; dashed where approximate, dotted where concealed, queried where questionable; arrow on fold axis denotes direction of plunge
49	Strike and Dip of Bedding
41	Approximate Bedding
24	Strike and Dip of Bedding Shear
C — C'	Geologic Section Line
B-1	Boring Location and Number
TP-9	Test Pit Location and Number
CS-3	Cut Slope Designation
	Subdrain

OFFSITE ZONE 1A WATER TANK SITE	
RAW CUT	87,000 CUBIC YARDS
RAW FILL	2,000 CUBIC YARDS
EXPORT	87,000 CUBIC YARDS

NEWHALL RANCH

100' 50' 0' 100' 200'
GRAPHIC SCALE
Note: For reduced sized prints, original scale is in inches

PROPOSED OFFSITE
ZONE 1A WATER TANK GRADING
EXHIBIT

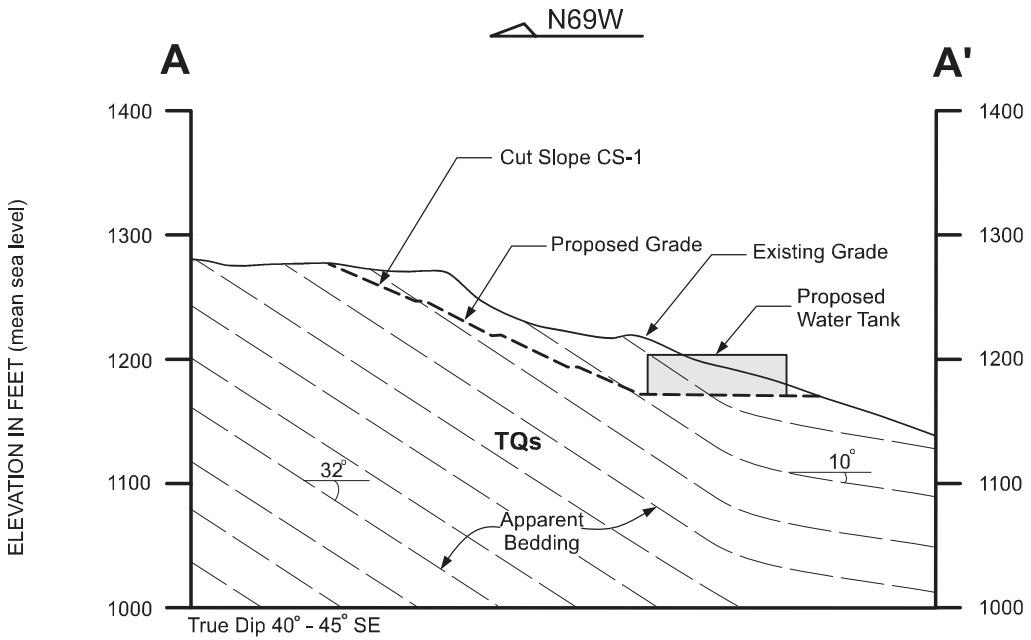
PSOMAS

DATE: 03-07-08 REVISED ON:
JOB No:1NRC010708

SHEET 1 OF 1
RV122-01

Plotted: 4/2/2008 10:45:03 AM :: Saved: 4/2/2008 10:44:25 AM :: C:\Documents and Settings\Mohammed\My Documents\DRAFTING\Data\1999\99802\3-28-08\RV122-01.dwg :: Mohammed

GEOLOGIC SECTION A-A'



Geologic Section

PREPARED FOR
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PROJECT LOCATION
Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	MN	
DATE	FIGURE 2.1	JOB NO.
3-28-08		99-802-21

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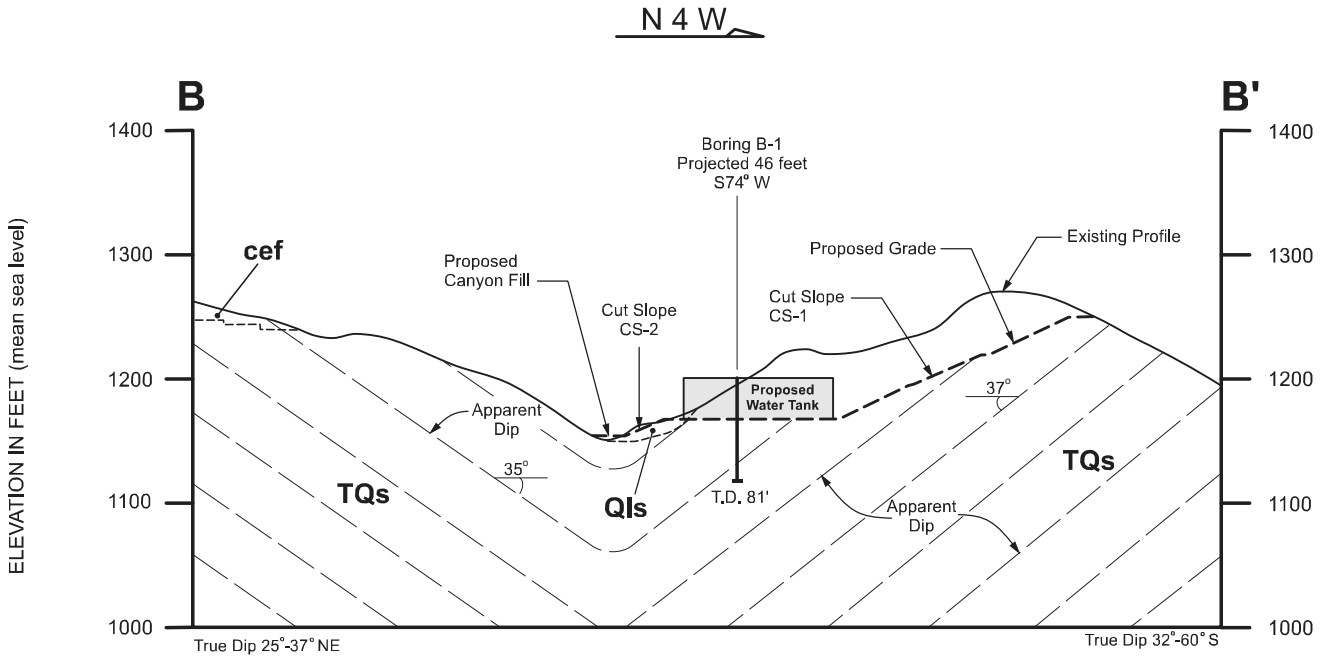
GEOLOGIC SECTION B-B'



Geologic Section

PREPARED FOR
Newhall Land and Farming Company

PROJECT LOCATION
Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	MN	
DATE	FIGURE 2.2	JOB NO.
3-28-08		99-802-21

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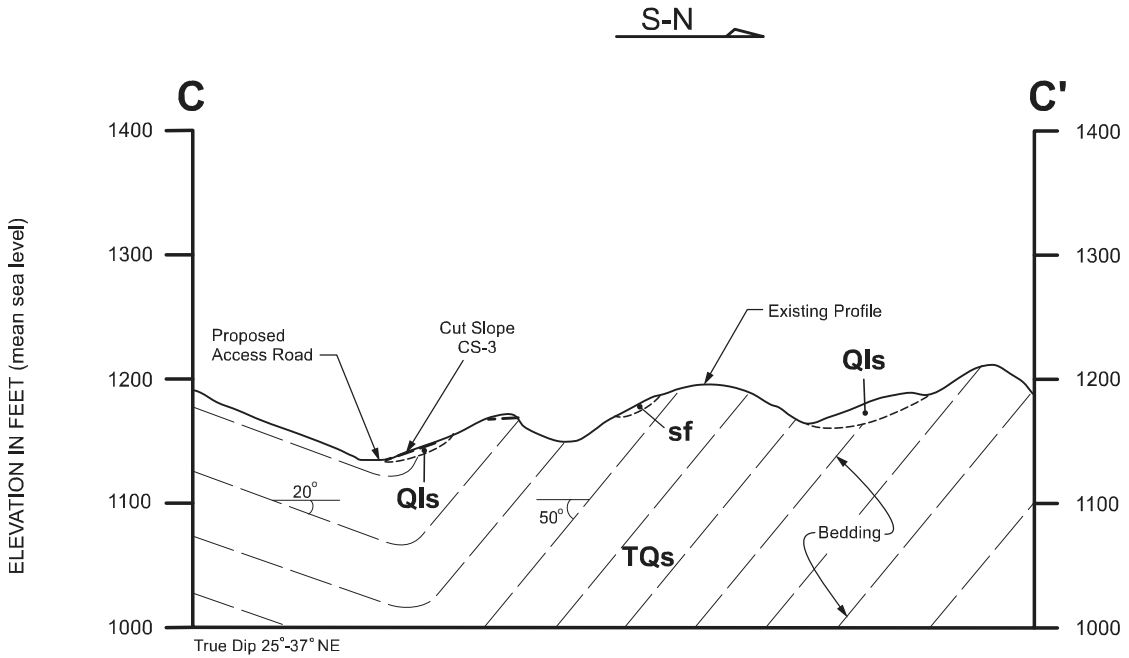
GEOLOGIC SECTION C-C'



Geologic Section

PREPARED FOR
Newhall Land and Farming Company

PROJECT LOCATION
Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	MN	
DATE	FIGURE 2.3	JOB NO.
3-28-08		99-802-21

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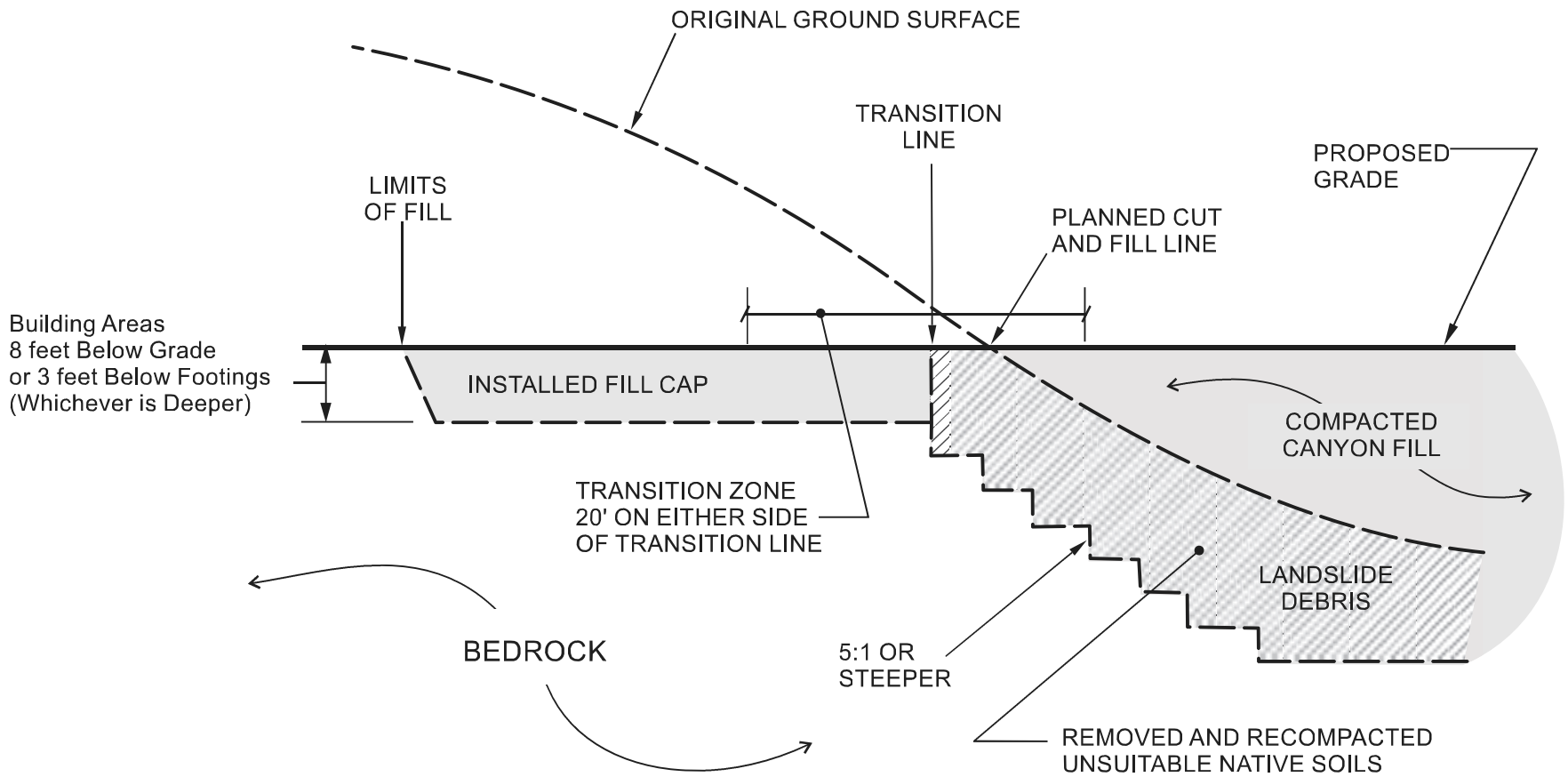
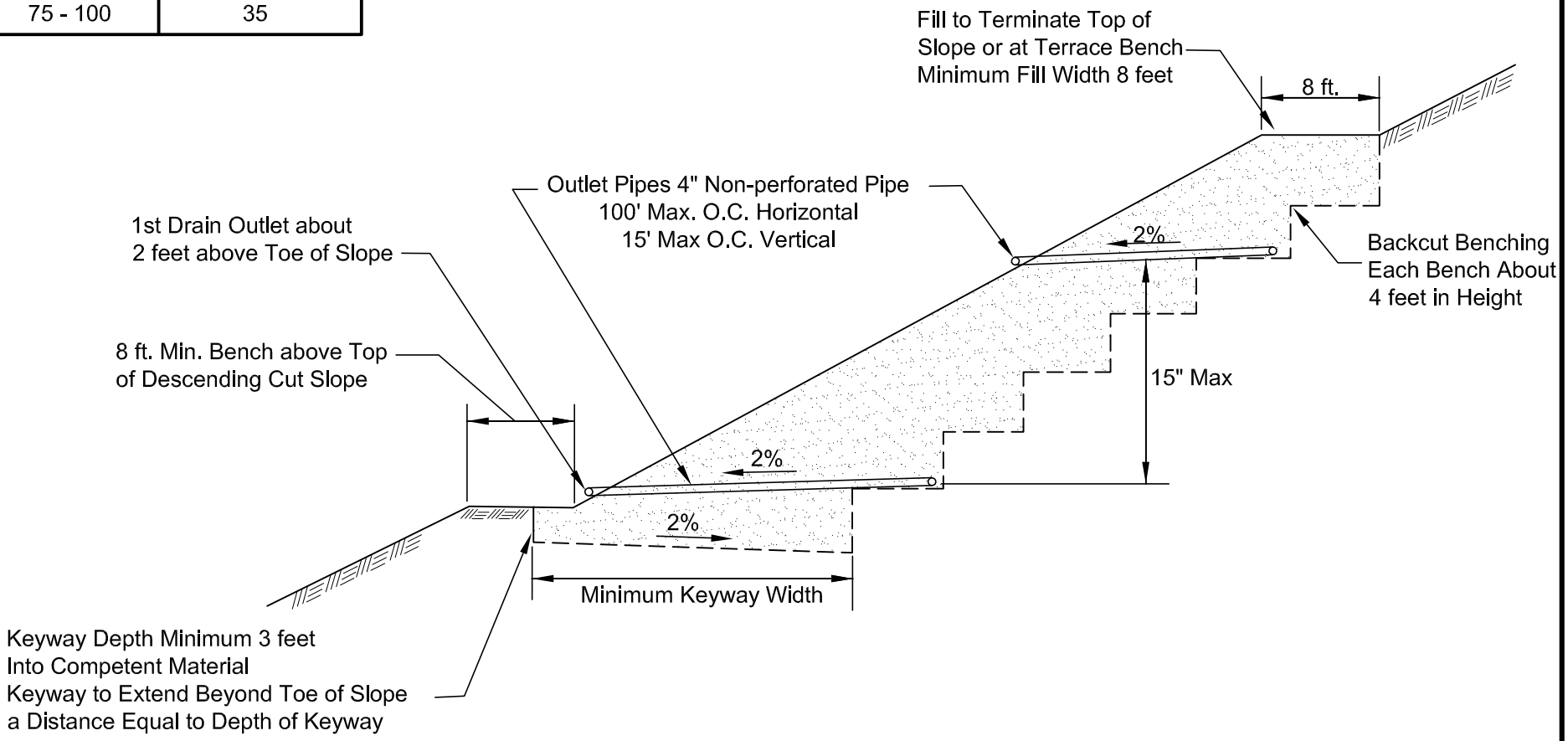


TRANSITION ZONE DETAIL

NOT TO SCALE

Stability Fill Height (feet)	Minimum Width of Keyway (feet)
0 - 20	15
20 - 30	20
30 - 50	25
50 - 75	30
75 - 100	35



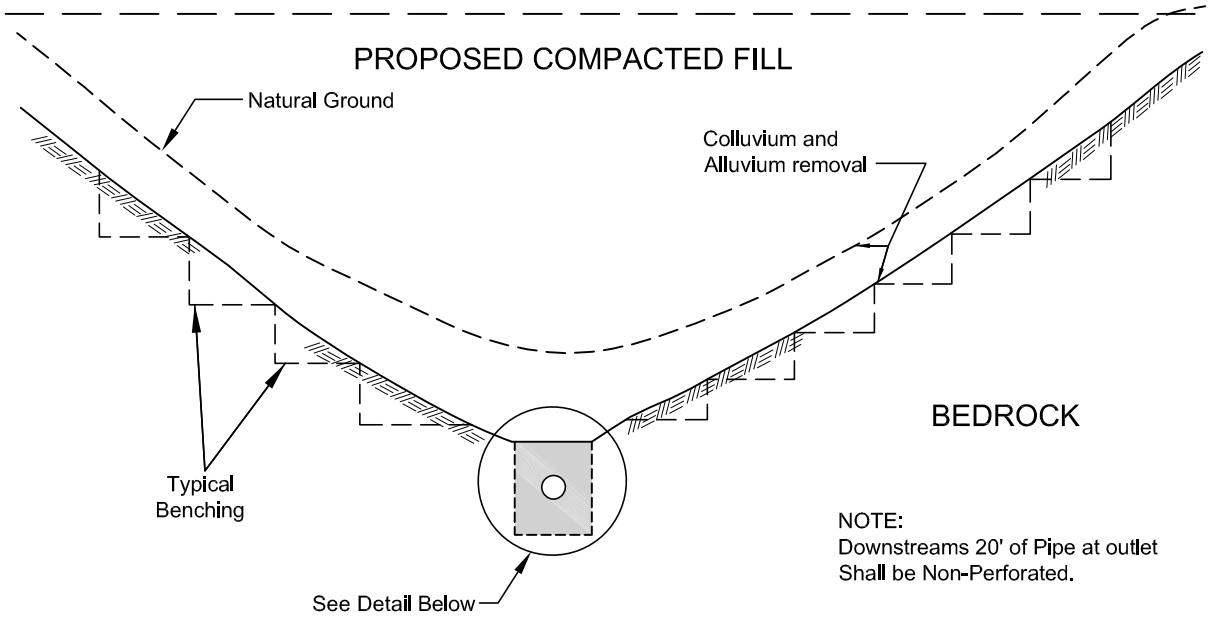
STABILITY FIIL DETAILS FOR GROSSLY STABLE SLOPES

N.T.S.

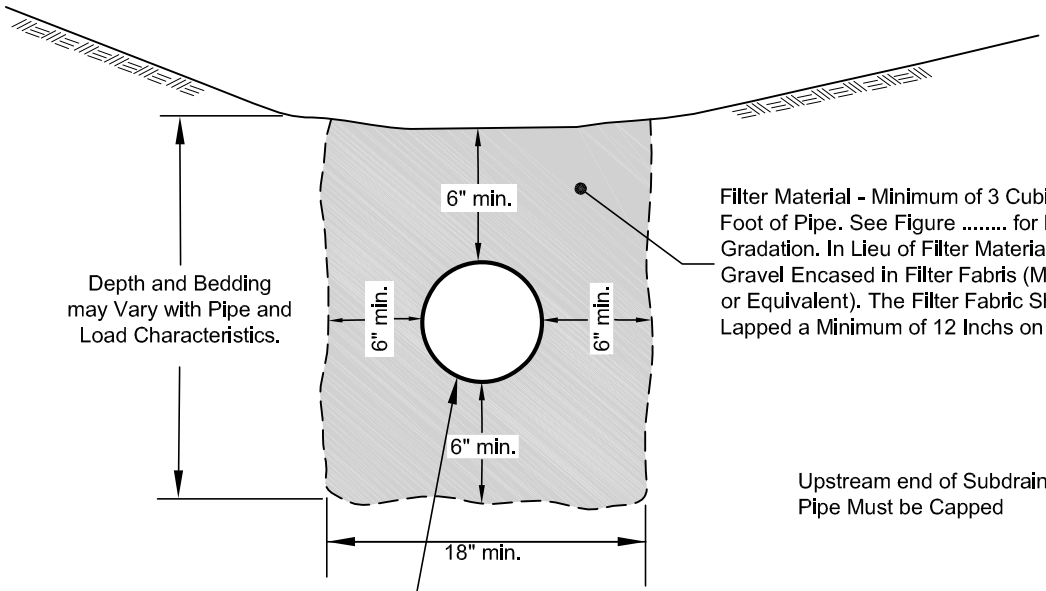
STABILITY(cad)

TYPICAL CANYON SUBDRAIN

MINIMUM STANDARD GRADING SPECIFICATIONS



NOTE:
Downstreams 20' of Pipe at outlet
Shall be Non-Perforated.



Filter Material - Minimum of 3 Cubic Feet per Foot of Pipe. See Figure ..... for Required Gradation. In Lieu of Filter Material Use Gravel Encased in Filter Fabris (Mirafi 140N or Equivalent). The Filter Fabric Shall be Lapped a Minimum of 12 Inchs on all Joints.

Upstream end of Subdrain
Pipe Must be Capped

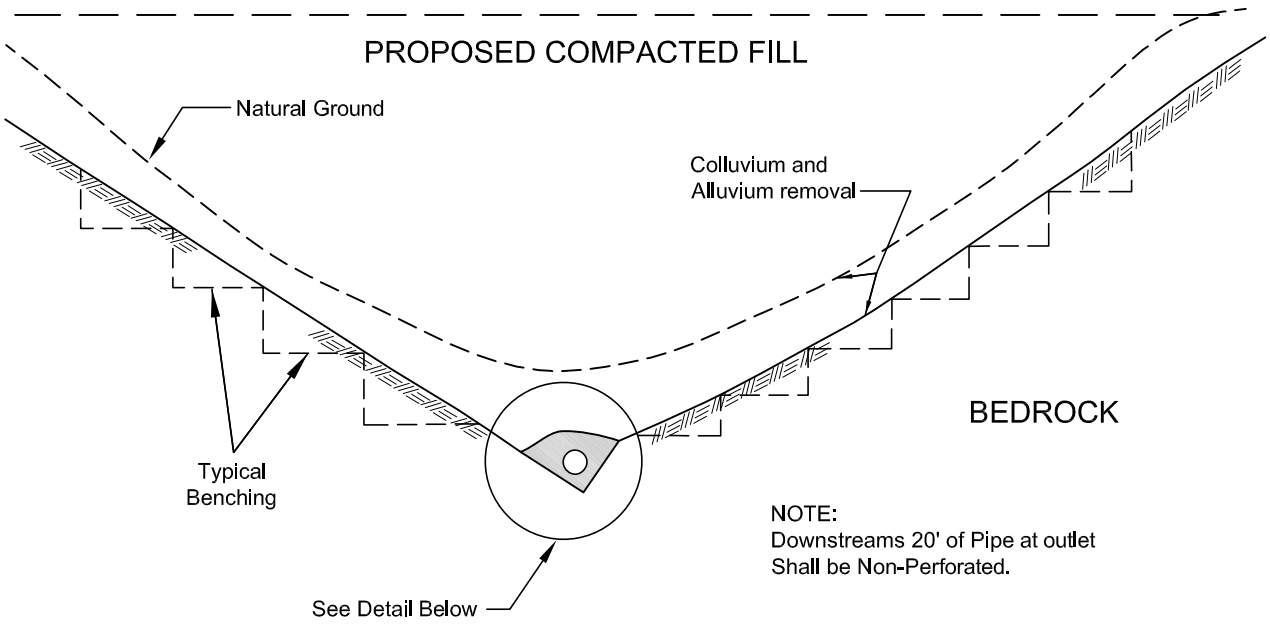
Minimum 4-inch Diameter, ADS Corrugated or ABS Class SDR 35 with a Crushing Strength of at Least 1000 Pound, with Minimum of 8 Uniformly Spaced Perforations per Foot of Pipe, Installed with Perforations on Bottom of Pipe.

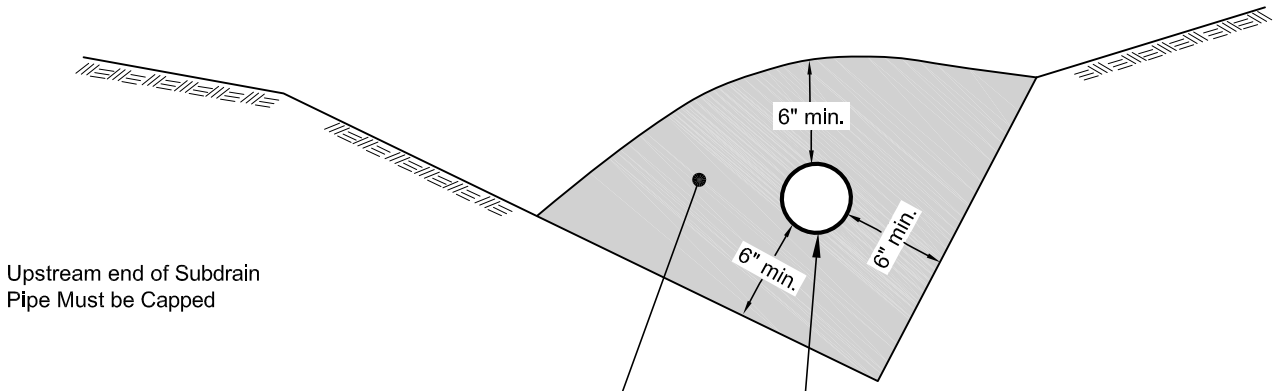
NOTE:
For Runs up to 500 feet Use 6-Inch Diameter Pipe
For Runs up to 500-1500 feet Use 8-Inch Diameter Pipe
For Runs Grather Than 1500 feet Use 10-Inch Diameter Pipe

CANYON-SUB(cad)

TYPICAL CANYON SUBDRAIN

MINIMUM STANDARD GRADING SPECIFICATIONS

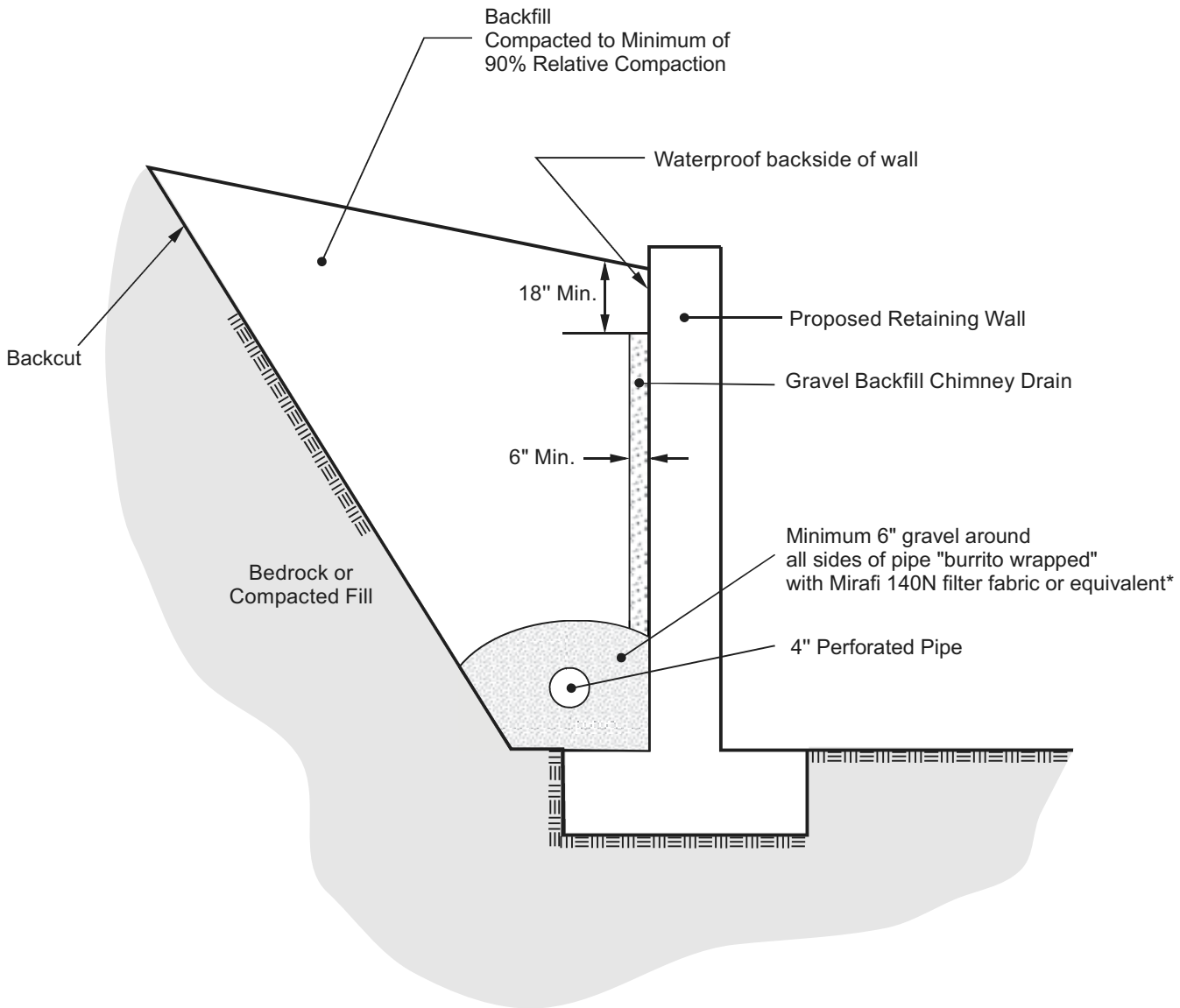




NOTE:
For Runs up to 500 feet Use 6-Inch Diameter Pipe
For Runs up to 500-1500 feet Use 8-Inch Diameter Pipe
For Runs Grater Than 1500 feet Use 10-Inch Diameter Pipe

RETAINING WALL DETAIL

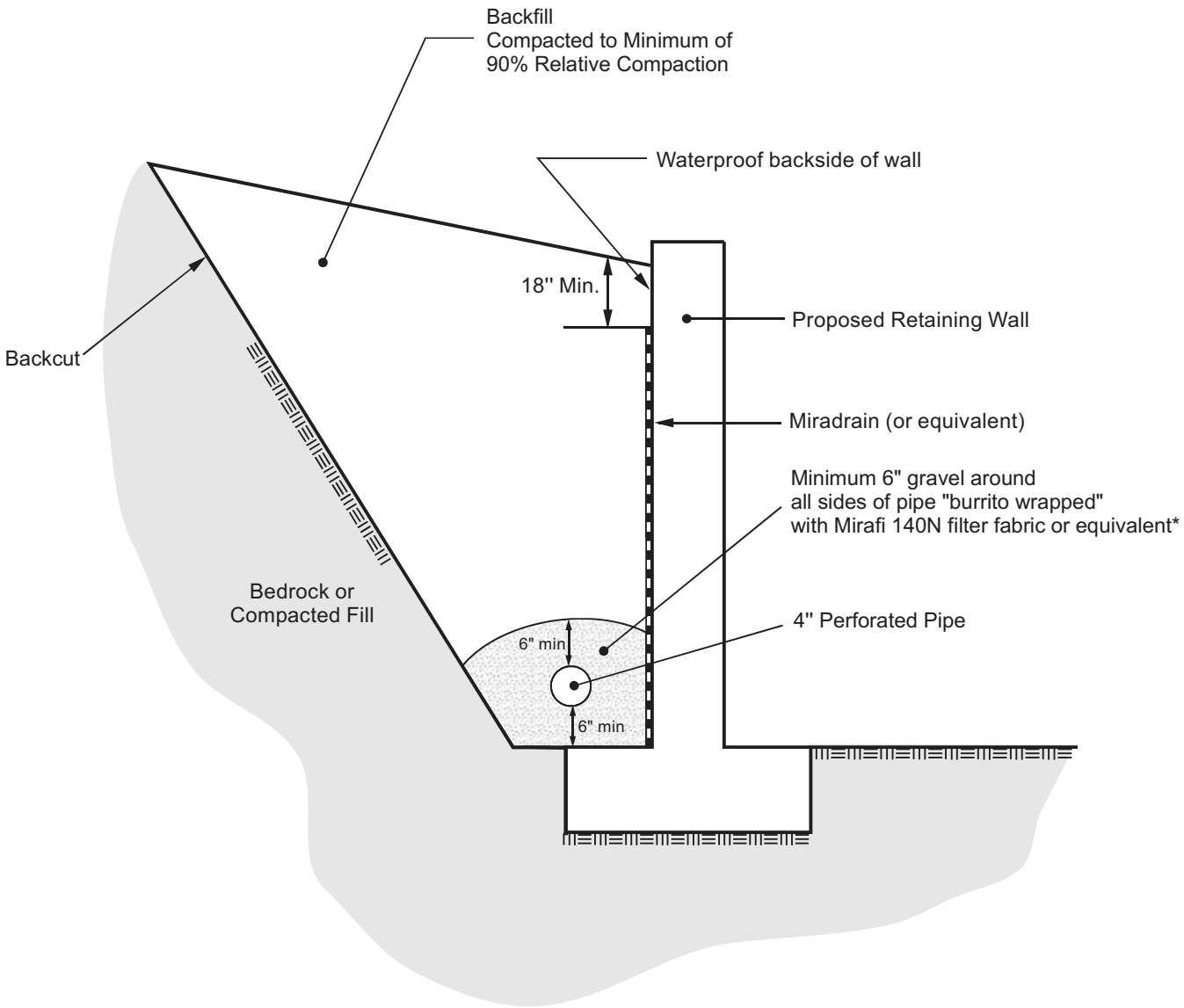
WITH CHIMNEY DRAIN



*F-1 Filter Material
may be used as an alternative
to gravel and filter fabric with
slotted pipe

RETAINING WALL DETAIL

WITH MIRADRAIN



*F-1 Filter Material
may be used as an alternative
to gravel and filter fabric with
slotted pipe

Newhall Land and Farming Company
March 28, 2008
99-802-21

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

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APPENDIX

FIELD EXPLORATIONS

The bedrock conditions beneath the site were explored in 2001 by drilling one bucket boring and excavation of test pits at the locations shown on the Geotechnical Map. The boring was drilled using a 24-inch-diameter bucket auger-type drilling equipment. The rock encountered was logged by our field geologist and undisturbed samples were obtained for laboratory inspection and testing. The lined-barrel sampler used to take undisturbed samples has an external diameter of 3.0 inches and an internal diameter of 2.625 inches. The depths at which the undisturbed samples were obtained are indicated on the logs. The sampler was driven using a hammer; the number of blows required to drive the sampler 12 inches are shown on the boring log. The driving weights are 3,160 pounds from 0 to 24 feet, 2,040 pounds from 24 feet to 47 feet, 1,120 pounds from 47 feet to 72 feet and 1,520 pounds from 72 feet to 81 feet.

LABORATORY TESTING

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties. The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left of the boring log.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the site materials. In addition, shear test data were also available from our prior investigations at and nearby the site. The tests were performed after soaking the samples to near-saturated moisture content and at various surcharge pressures. The yield strength values determined from the direct shear tests are presented on Summary of Shear Test Data.

The optimum moisture content and maximum dry density of the existing fill soils were determined by performing compaction tests on a bulk sample bag obtained

from the boring. The test was performed in accordance with the ASTM Designation D1557-02 method of compaction. The results of the tests are presented below.

Soil Description and Classification	Maximum Dry Density (lbs./cu. ft.)	Optimum Moisture Content (%)
Light gray brown fine to medium sand (SP)	131	8
Light brown silty fine to medium sand (SM)	126	9.5

The optimum moisture contents are indicated in percent of dry weight, and the maximum dry densities are in pounds per cubic foot (pcf). The double-letter soil classifications that follow the Soil Descriptions are in accordance with the Uniform Soil Classification System (ASTM D 2487-00).

BORING B-1

JOB NUMBER: 99-802-01
DATE DRILLED: 6-4-01
EQUIPMENT USED: 24"-Diameter Bucket Auger
ELEVATION: 1200 +/-
SURFACE CONDITIONS: Graded Pad

BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	SOIL TYPE
17	3	120	-	5		SAUGUS FORMATION (TQs) SANDSTONE, very fine to fine, slight iron-oxide staining, soft, dry, yellowish orange to light gray 1" thick clayey silt bed: N56E, 56SE @ 2.5 feet: fine to coarse, some laminations
17	4	126	-	10		SANDY SILTSTONE to SILTY SANDSTONE, very fine very soft, slightly moist, light brownish gray @ 6.5 feet: Bedding-N60E, 45SE mafic mineral laminations: Bedding-N65E, 60SE
15	5	122	-	15		PEBBLY SANDSTONE, fine to very coarse with pebbles and gravel, slightly clayey in upper 6 inches, soft, slightly moist, yellowish gray @ 12.5 feet: 1/8" pale red clay shear: N72E, 24SE; truncates laminations
-	-	-	-	20		CONGLOMERATIC SANDSTONE, very coarse, abundant pebbles, small cobbles, soft, slightly moist, moderate yellowish brown @ 17 feet: 1" thick grayish brown clay seam (possible bedding shear): N55E, 50SE Bedding-N73E, 43S
25	4	122	-	25		Interbedded PEBBLY SANDSTONE and CONGLOMERATIC SANDSTONE, coarse to very coarse with abundant pebbles and small cobbles, soft, slightly moist, pale yellowish brown to moderate yellowish brown, average bed thickness 1½ to 2 feet
24	11	124	-	30		approximate bedding-N70E, 44SE SILTY CLAYSTONE, very soft, moist, moderate reddish brown, soft sediment deformation at contact with overlaying SANDSTONE
22	5	123	-	35		SANDSTONE, very fine to fine, finely laminated with laminations defined by mafic minerals, soft, slightly moist, moderate yellowish brown to yellowish gray Bedding-N52E, 44SE fine to coarse Bedding-N54E, 45SE
				40		

Note: The log of subsurface conditions shown hereon applies only at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations or times.

(CONTINUED ON THE FOLLOWING FIGURE)

LOG OF BORING

R.T. FRANKIAN & ASSOCIATES

BORING B-1 (continued)

JOB NUMBER: 99-802-01
DATE DRILLED: 6-4-01

BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (LBS. PER CU. FT.)	N-VALUE	DEPTH (FEET)	SAMPLE LOCATION	SOIL TYPE
30	4	123	-	45		SANDSTONE, very coarse, soft, slightly moist, yellowish gray
20	5	119	-			trace cobbles
25	5	112	-	50		SANDSTONE to PEBBLY SANDSTONE, very coarse with pebbles, soft, slightly moist, yellowish gray to very pale orange
38	-	-	-	55		CONGLOMERATE, very coarse sand matrix, abundant gravel and small cobbles, massive, soft, slightly moist, yellowish gray to pale yellowish brown
						Bedding-N62E, 43SE
17	9	119	-	60		SANDSTONE to PEBBLY SANDSTONE, fine to very coarse, massive soft, slightly moist, pale yellowish brown
						1" thick brownish gray clay seam (possible bedding shear): N68E, 43SE
22	4	112	-	65		SANDSTONE, fine to very coarse, soft, slightly moist, pale yellowish brown
						some cross bedding
29	4	113	-	70		
						Bedding-N86W, 43SW
				75		PEBBLY SANDSTONE, coarse to very coarse, abundant pebbles and gravel, soft, slightly moist, pale yellowish brown to black yellowish orange
						END OF BORING AT 81 FEET. No water. No caving.
20	8	112	-	80		slightly silty

Note: The log of subsurface conditions shown hereon applies only at the specific location and date indicated. It is not warranted to be representative of subsurface conditions at other locations or times.

LOG OF BORING

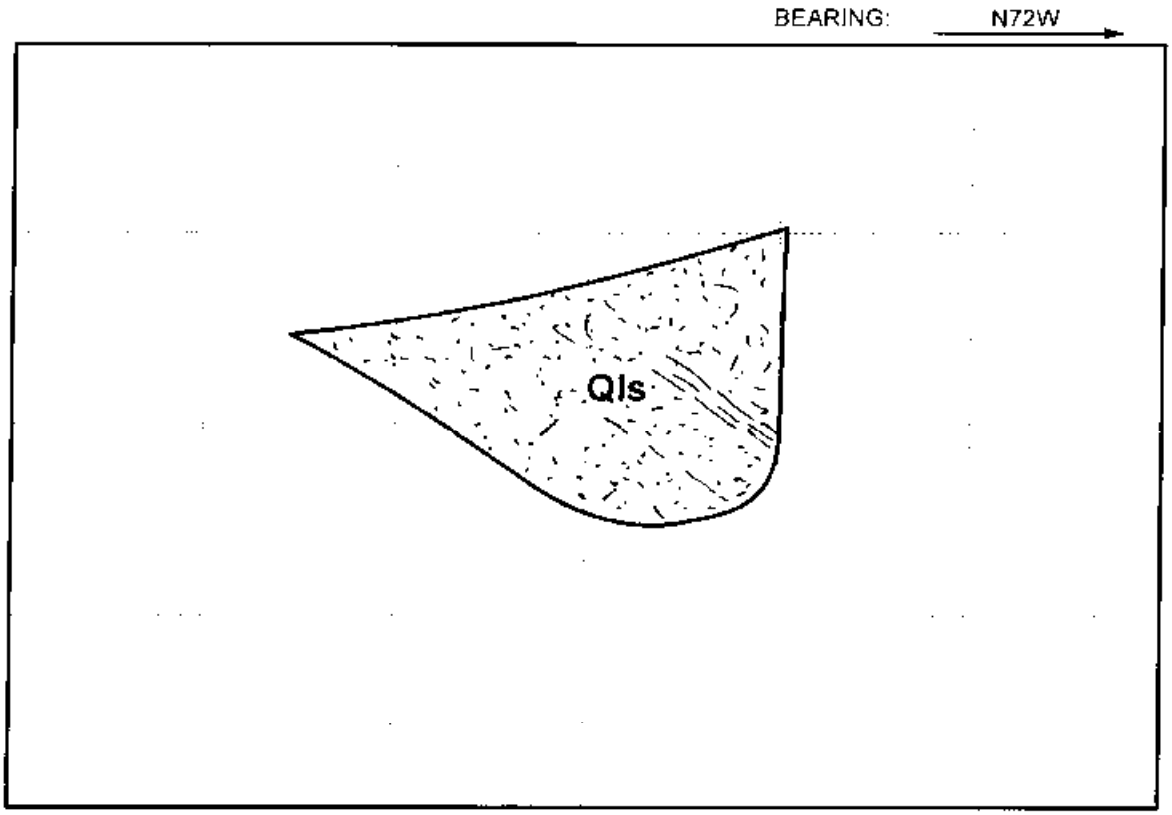
R.T. FRANKIAN & ASSOCIATES

LOG OF TEST PIT TP-1

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1090' DATE LOGGED 5-15-01

0'-7' **LANDSLIDE (Qls):** fine grained sand with discontinued beds of siltstone, soft, dry, Pale yellowish brown (10YR 6/2)



SCALE: 1" = 5'

NOTE
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

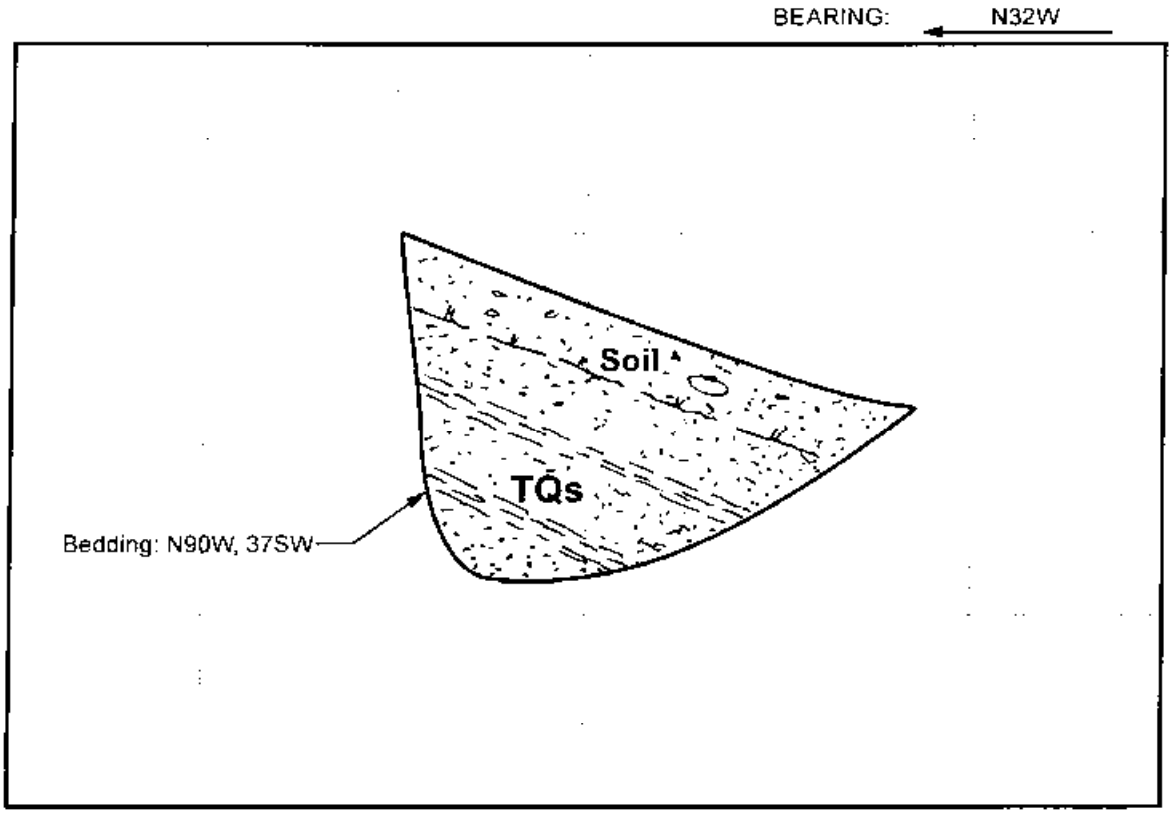
LOG OF TEST PIT TP-2

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1070' DATE LOGGED 5-15-01

0'-2' **SOIL / COLLUVIUM (Qc):** fine to coarse grained silty sand with sandstone cobbles, soft, dry, pale yellowish brown (10YR 6/2)

2'-9' **SAUGUS FORMATION (TQs):** fine to coarse grained sandstone, hard, moist, pinkish gray (5YR 8/1), clayey siltstone, moderately hard, moist, moderate brown (5YR 4/4)



SCALE: 1" = 5'

NOTE:
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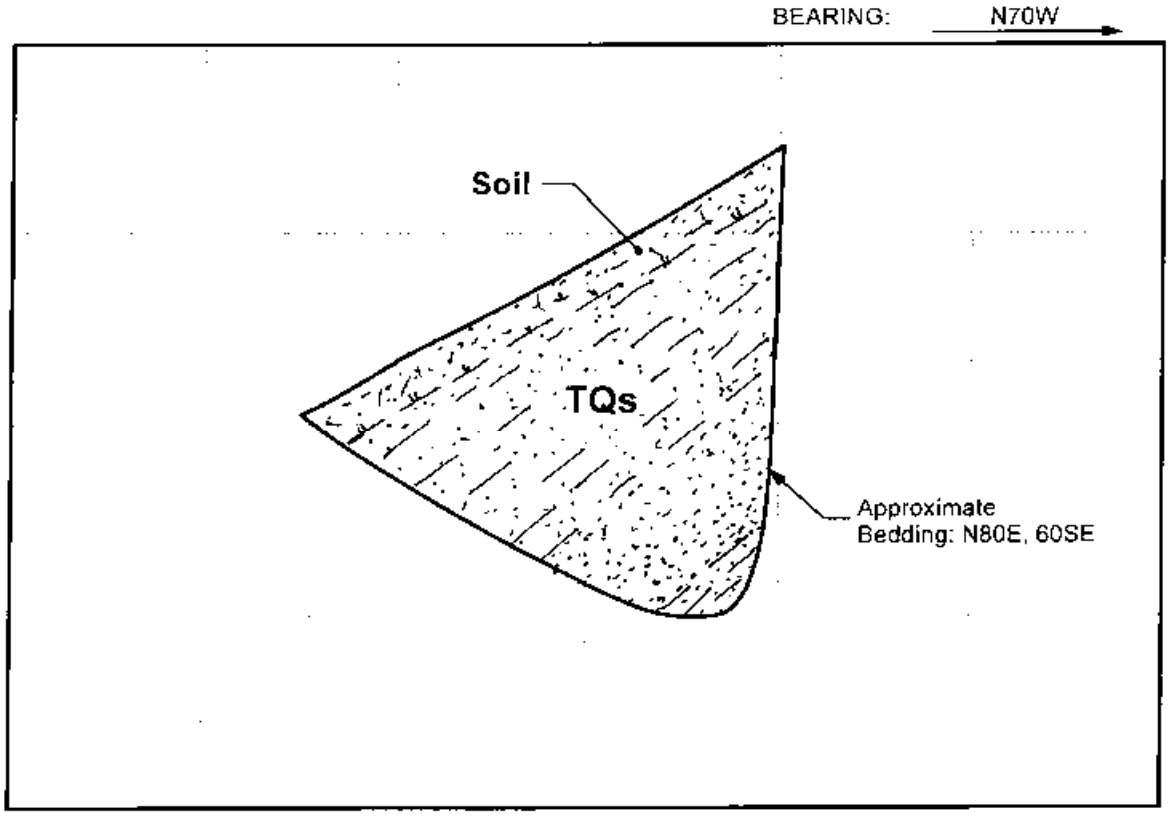
LOG OF TEST PIT TP-3

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1105' DATE LOGGED 5-15-01

0'-2' **SOIL:** fine to coarse grained silty sand to silt, loose, dry, pale yellowish brown (10YR 6/2)

1'-12' **SAUGUS FORMATION (TQs):** fine grained sandy siltstone, moderately hard, moist, moderate yellowish brown (10YR 5/4),random fractures, no well define bedding



SCALE: 1" = 5'

NOTE:
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

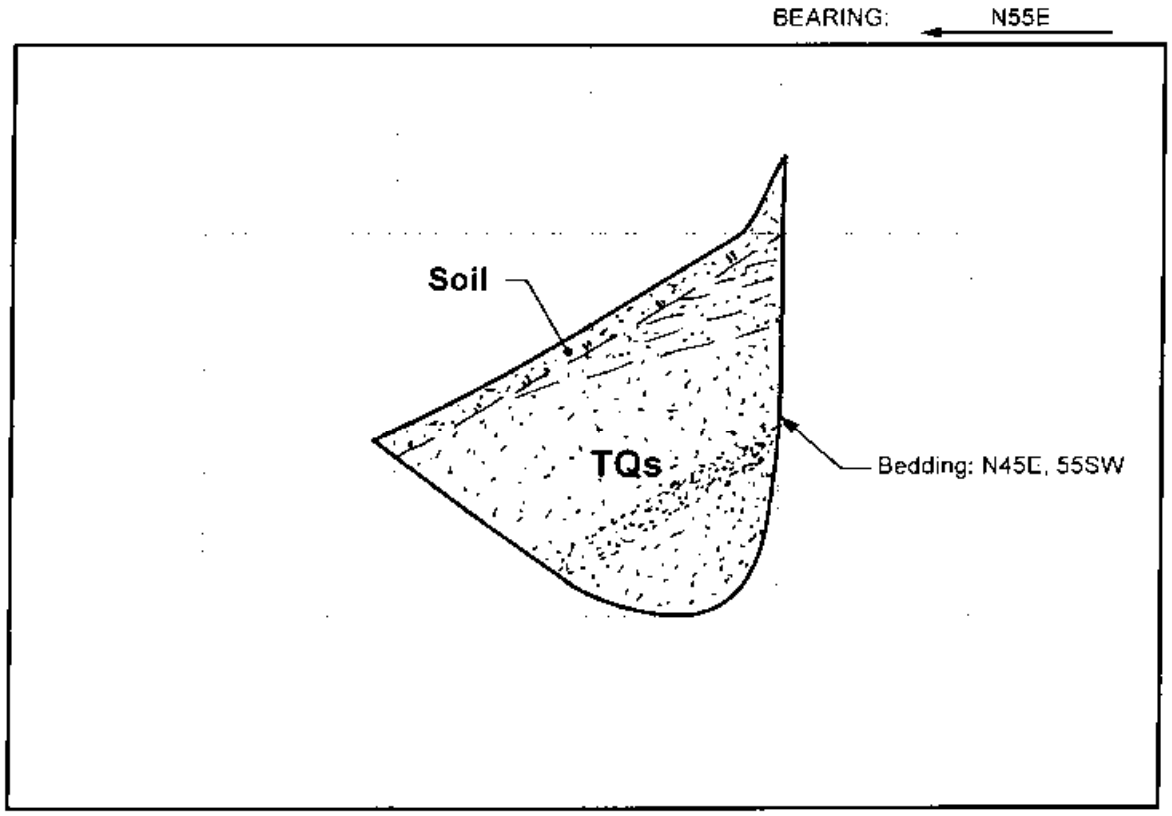
LOG OF TEST PIT TP-4

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1080' DATE LOGGED 5-15-01

0'-0.5' **SOIL:** mostly sand, some silt, pale yellowish brown (10YR 6/2),loose, dry

2'-9' **SAUGUS FORMATION (TQs):** fine to medium grained sandstone, moderately hard, dry, pinkish gray (5YR 8/1) , well bedded



SCALE: 1" = 5'

NOTE:
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

R.T. FRANKIAN & ASSOCIATES

LOG OF TEST PIT TP-5

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1135' DATE LOGGED 5-15-01

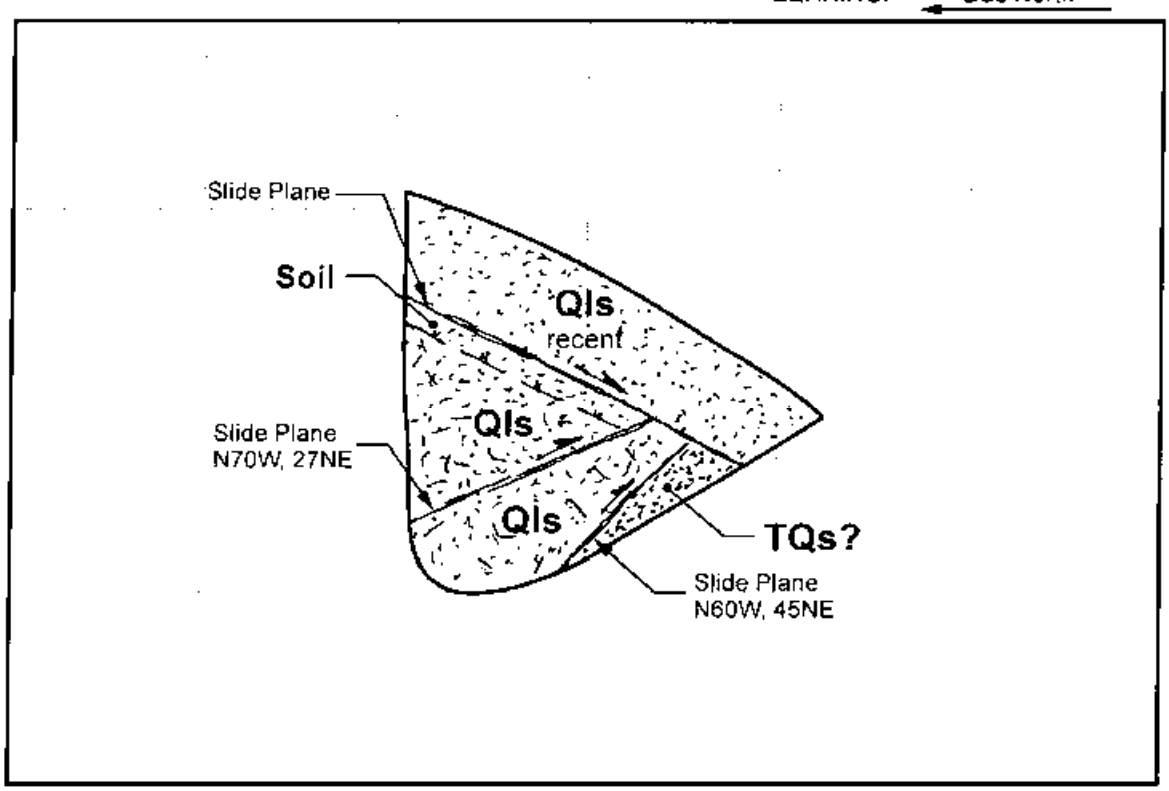
0'-10' **LANDSLIDE (Qls):**

0'-2' fine grained silty sand, loose, dry, grayish orange (10YR 7/4)

2'-2.5' **SOIL**

4'-10' silty sandstone, moderately hard, dry to moist, pale yellowish brown (10YR 6/2), fractured

BEARING: Due North



SCALE: 1" = 5'

NOTE:
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

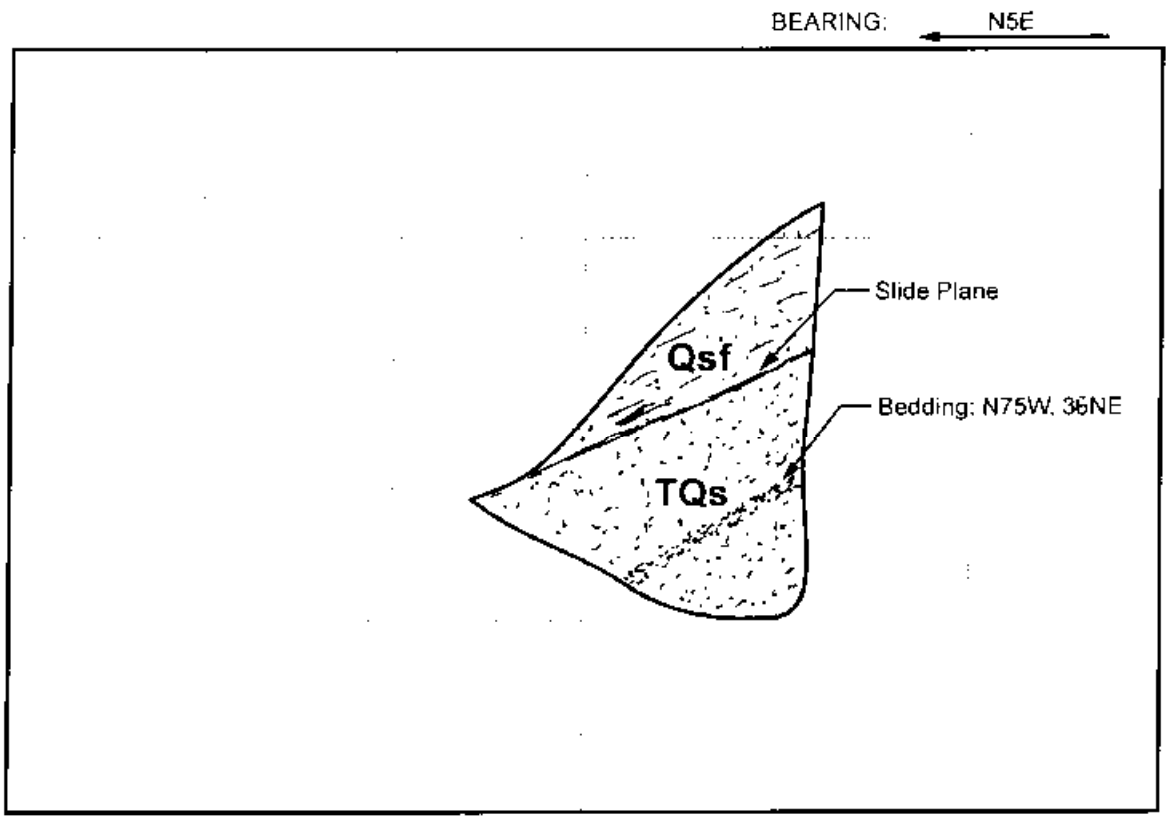
LOG OF TEST PIT TP-6

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1140' DATE LOGGED 5-15-01

0'-4' **SURFICIAL FAILURE (Qsf)**: clayey siltstone moderately hard, moist, dark yellowish brown (10YR 4/2)

4'-11' **SAUGUS FORMATION (TQs):** fine grained silty sand, hard, moist, grayish orange (10YR 7/4)



SCALE: 1" = 5'

NOTE:
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

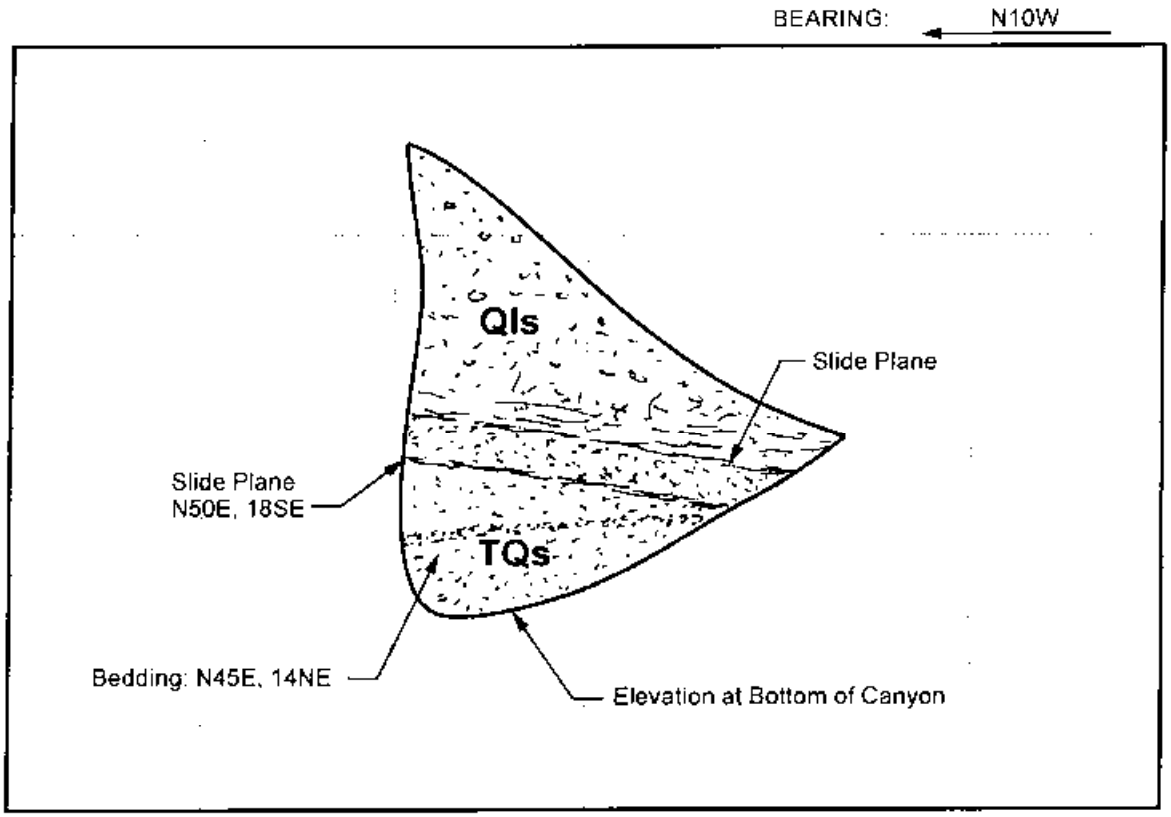
LOG OF TEST PIT TP-7

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1155' DATE LOGGED 5-15-01

0'-7' **LANDSLIDE (Qls):** fine grained silty sand with beds and scattered cobbles, loose, dry, grayish orange (10YR 7/4)

7'-12' **SAUGUS FORMATION (TQs):** sandy siltstone, moderately hard, moist, pale yellowish brown (10YR 6/2), with coarse grained sandstone bed



SCALE: 1" = 5'

NOTE
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

R.T. FRANKIAN & ASSOCIATES

For use with report dated 03/28/2008; RTF Job No. 99-802-21

LOG OF TEST PIT TP-8

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

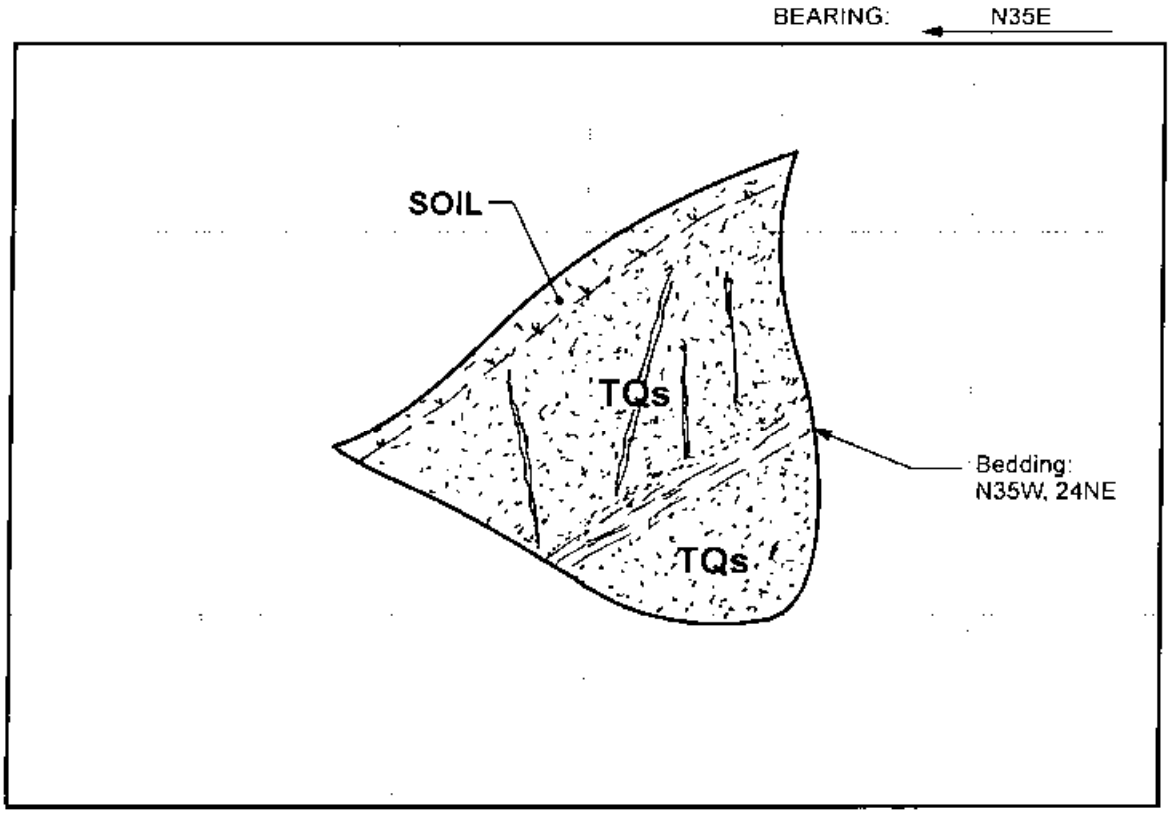
LOCATION P.M. 18108 ELEVATION 1160' DATE LOGGED 5-15-01

0'-0.5' **SOIL:** fine to coarse grained sand, loose, dry, pale yellowish brown (10YR 6/2)

0.5'-12.5' **SAUGUS FORMATION (TQs):**

0.5'-6' fine to coarse grained sandstone, loose, moist, pinkish gray (5YR 8/1), highly weathered, moderately well defined bedding at 6 feet

6'-12.5' fine grained silty sand, moderately hard, moist, grayish orange (10YR 7/4), with siltstone interbed



SCALE: 1" = 5'

NOTE
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES

R.T. FRANKIAN & ASSOCIATES

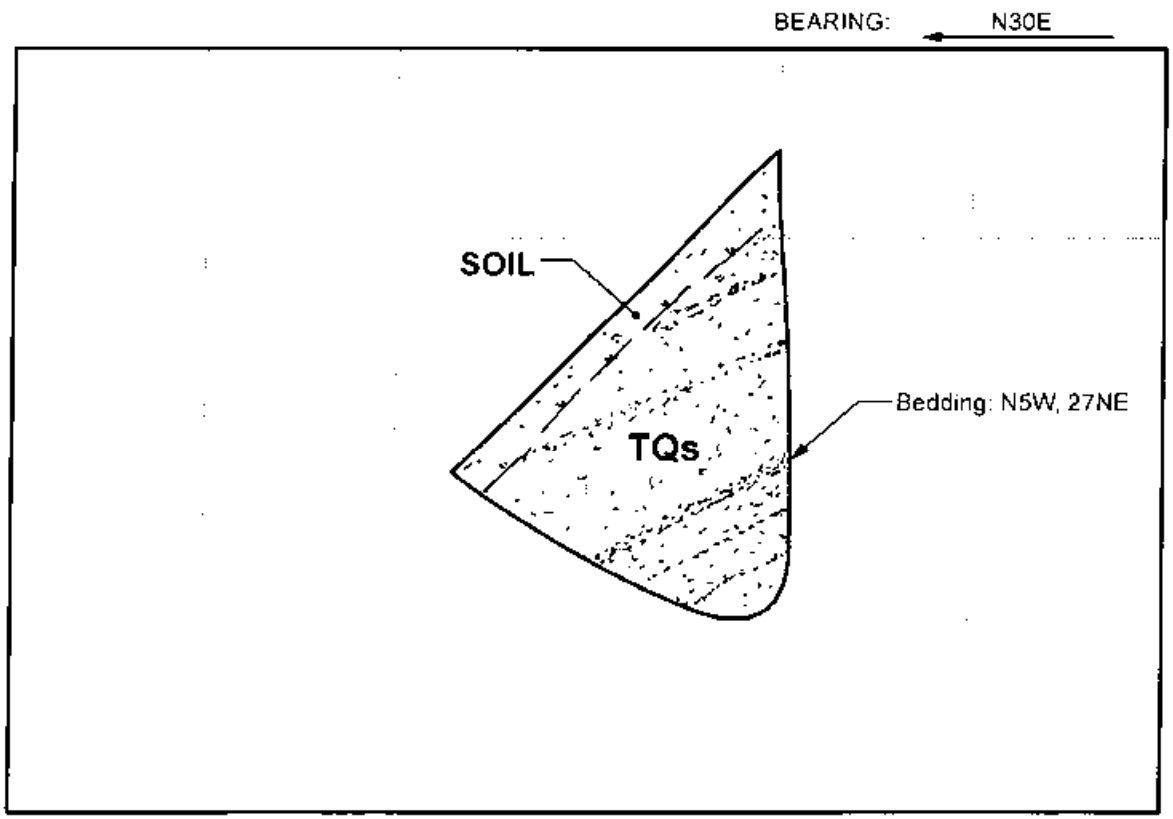
LOG OF TEST PIT TP-9

JOB NUMBER 99-802-01 CLIENT VALENCIA WATER COMPANY LOGGED BY DGF

LOCATION P.M. 18108 ELEVATION 1170' DATE LOGGED 5-15-01

0'-1' **SOIL**: fine to coarse grained sand with silt, loose, dry, pale brown (5YR 5/2),

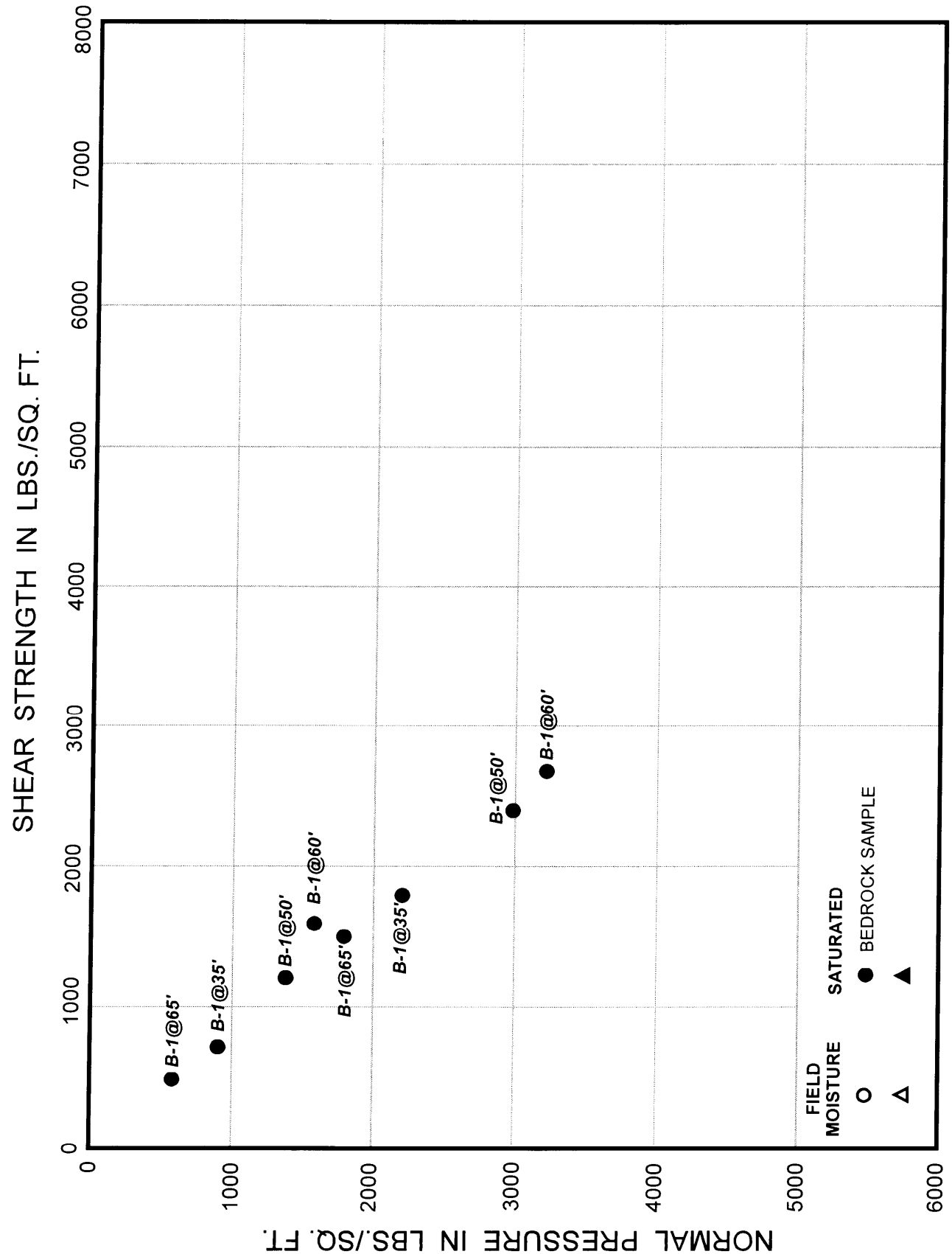
1'-13' **SAUGUS FORMATION (TQs):** fine to coarse grained sandstone with pebble conglomerate interbeds, soft, moist, pinkish gray (5YR 8/1), poorly cemented, well bedded, rock very poorly cemented possibly disturbed or creep affected



SCALE: 1" = 5'

NOTE:
THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC TEST PIT LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

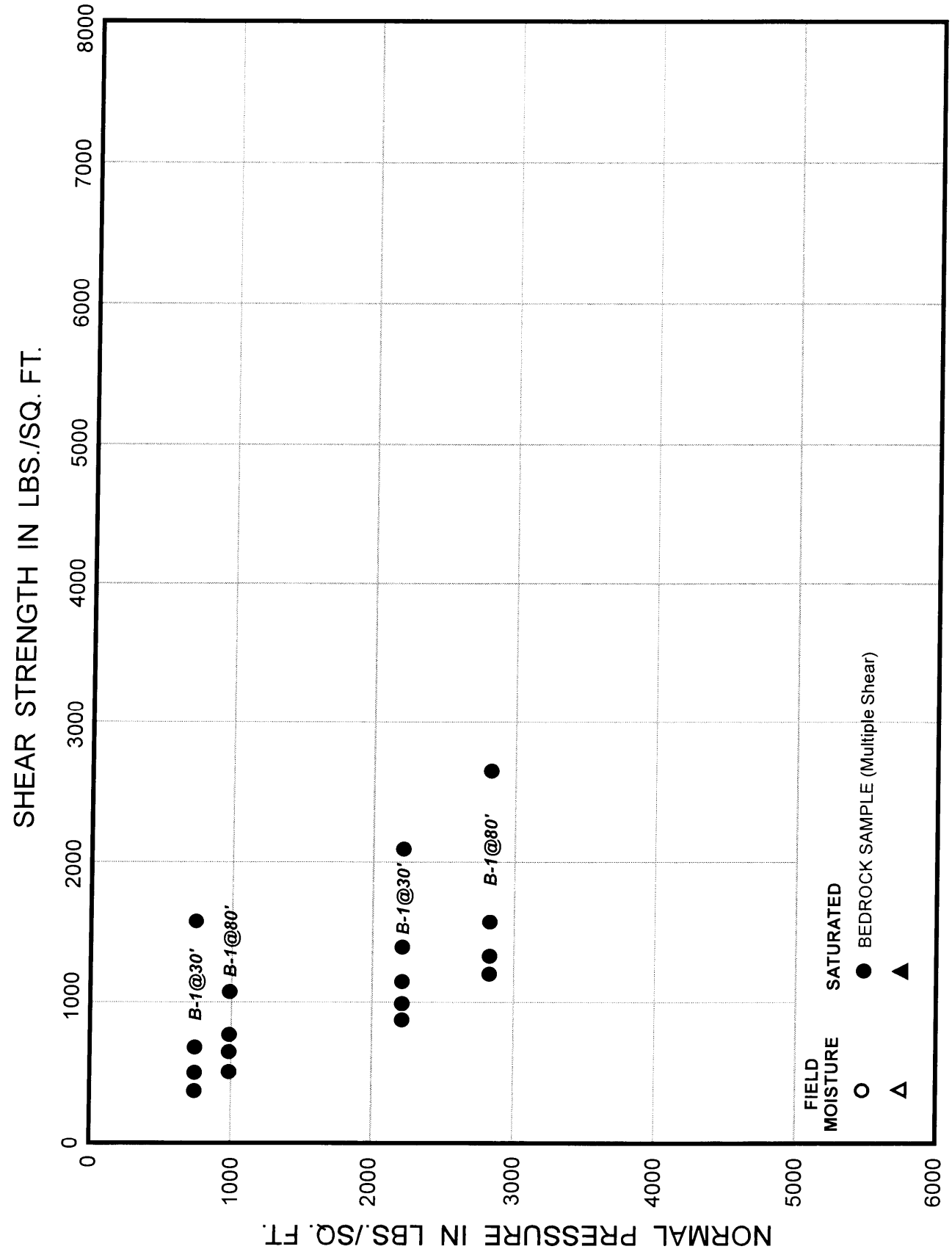
SUMMARY OF SHEAR TEST DATA



R.T. FRANKIAN & ASSOCIATES

FIGURE A-1a

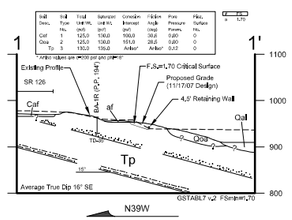
SUMMARY OF SHEAR TEST DATA

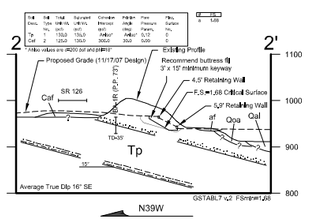


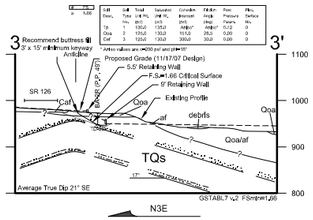
R.T. FRANKIAN & ASSOCIATES

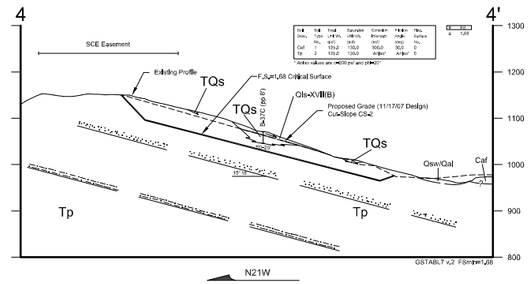
FIGURE A-1b

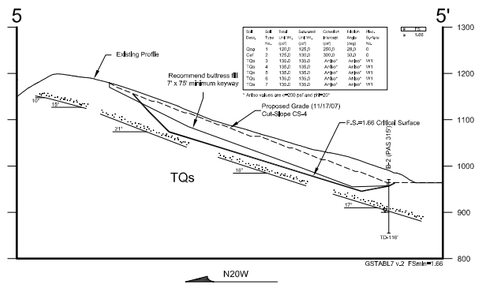
Appendix E

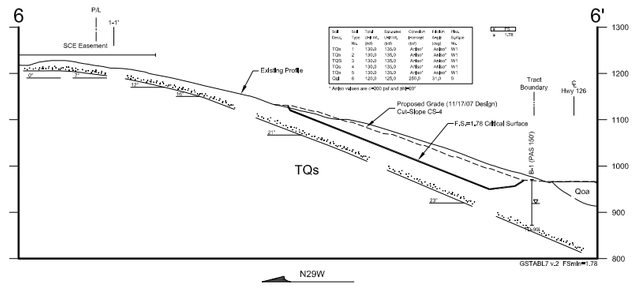


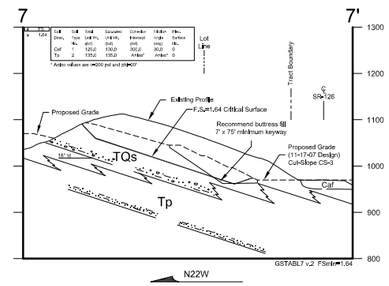


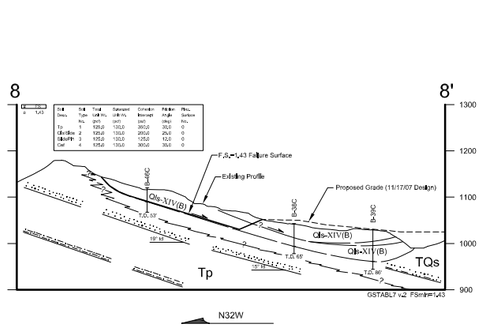


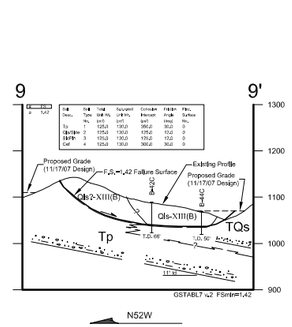


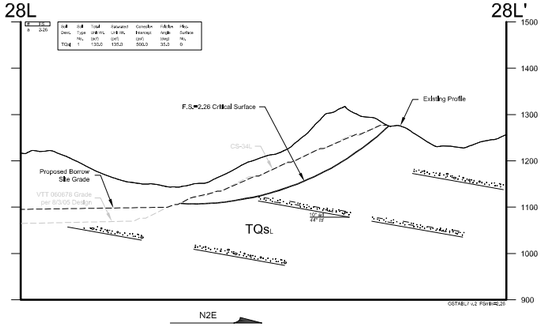


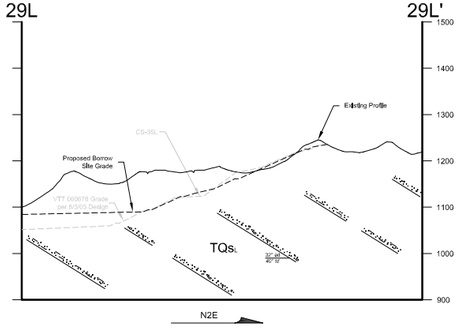


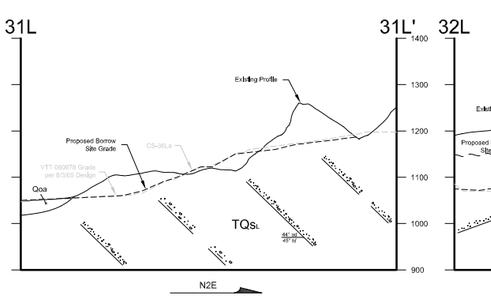


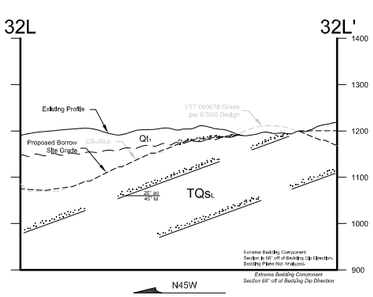


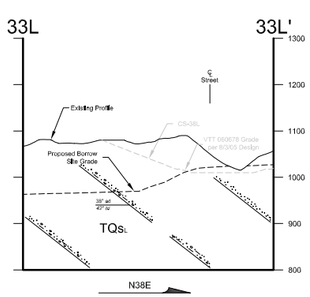


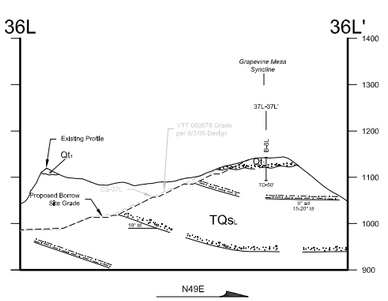


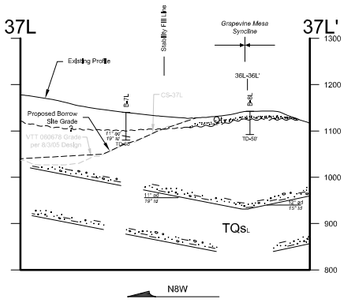








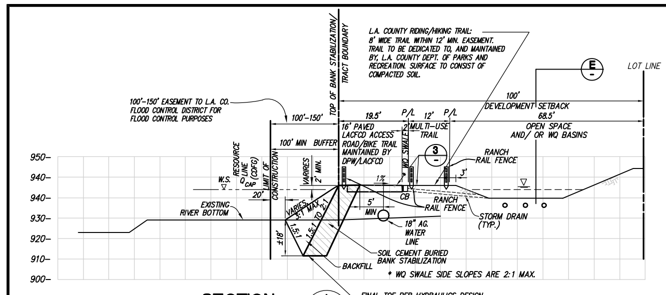




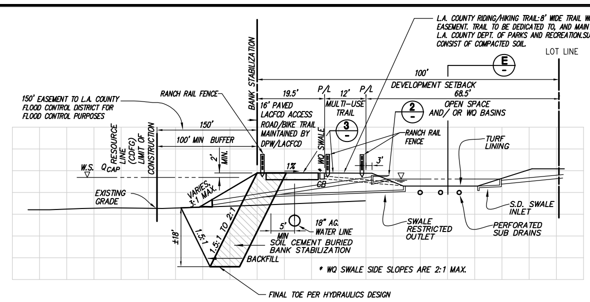
ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

GEOLOGIC CROSS SECTIONS

Date: 12/21/09	Job No.: 09-1702R
Scale: 1" = 100'	CAD File: Revised Sections Plate
PLATE I	Geology by: staff / Drawn by: staff
	Revised:



SECTION (A)
NOT TO SCALE



SECTION (B)
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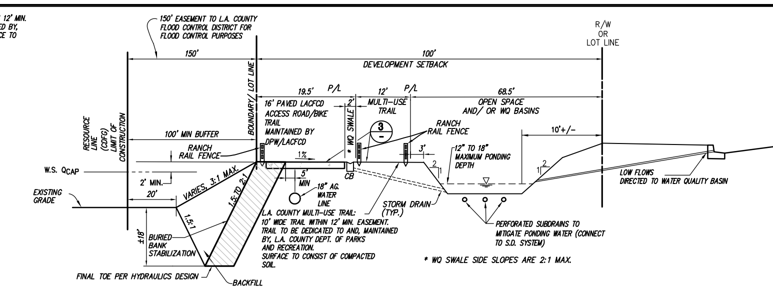
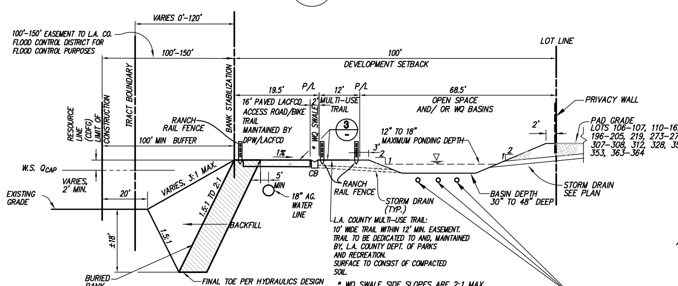
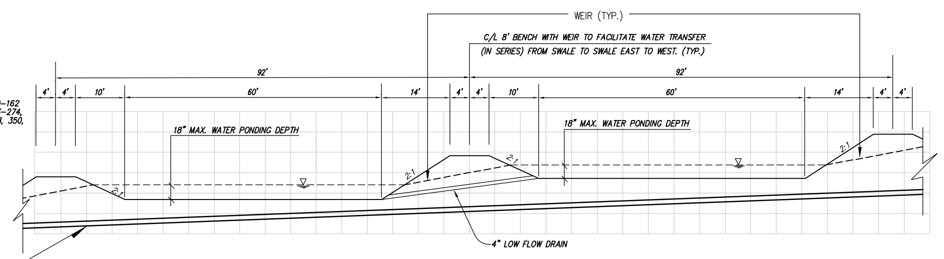


WATER QUALITY BASIN AT LOT 311 (OPEN SPACE) (1)

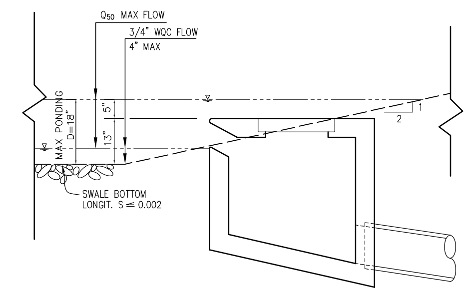


TYPICAL WATER QUALITY SWALE CROSS-SECTION * N.T.S.
*OPEN SPACE LOTS (ADJACENT TO LOTS 106-107, 110-162, 196-205, 219, 273-274, 307-308, 312, 328, 350, 353, 363-364)

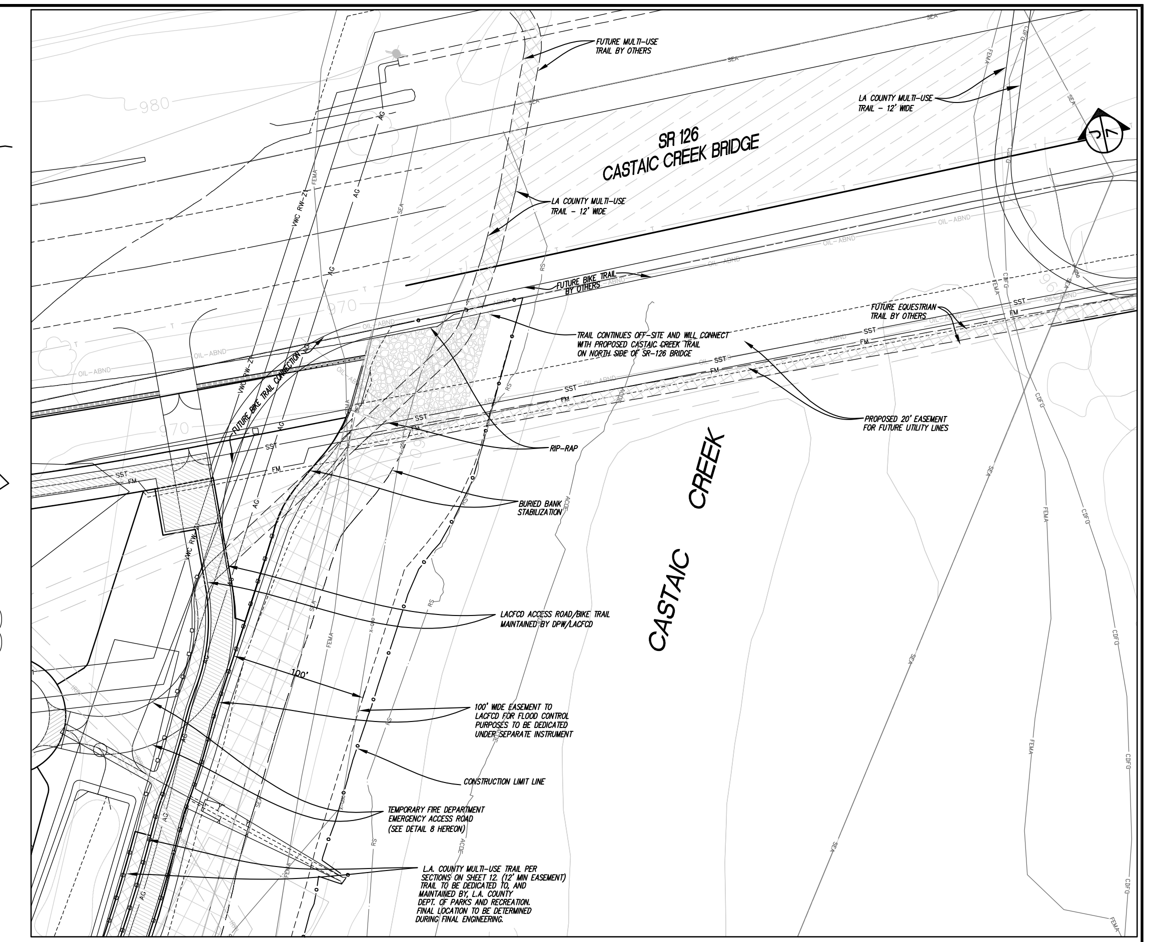


PROPOSED WATER QUALITY SWALE
LONGITUDINAL SECTION LOOKING NORTH N.T.S.

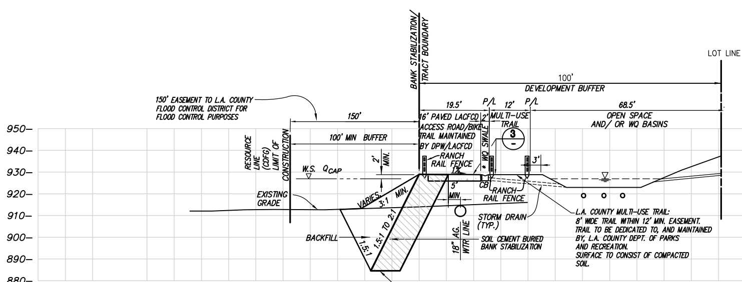
PROPOSED WATER QUALITY SWALE DETAILS (E)
NOT TO SCALE



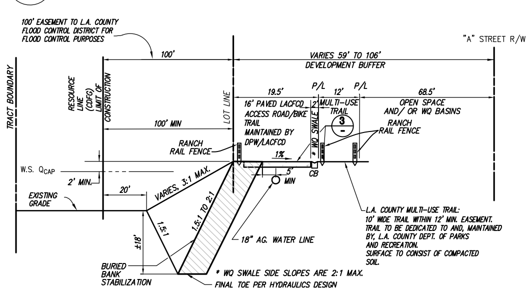
DRAINAGE RELIEF ON WQC SWALES (2)



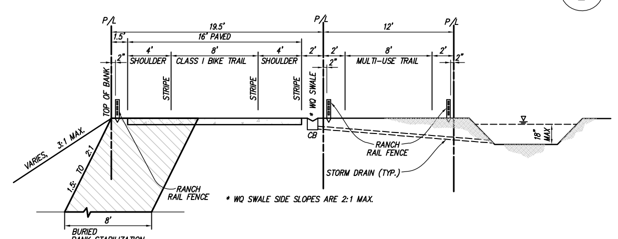
ACCESS ROAD AT CASTAIC CREEK BRIDGE (7)
SCALE: 1" = 40'



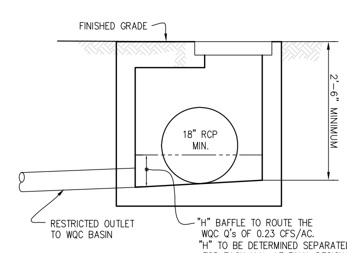
SECTION (F)
NOT TO SCALE



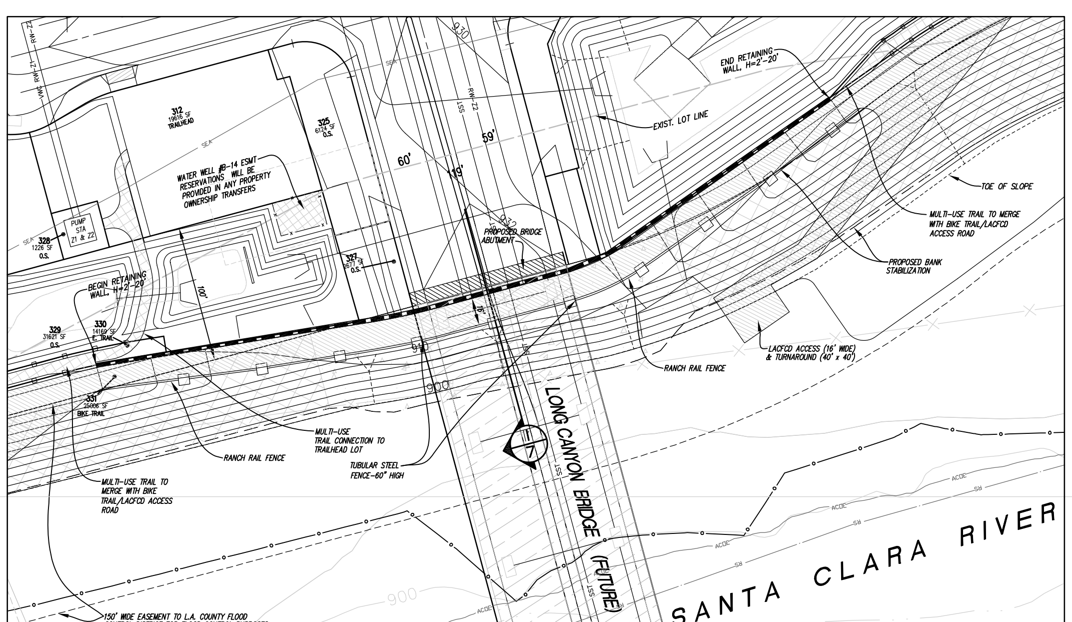
LOT 372 (O.S. LOT) (G)
NOT TO SCALE



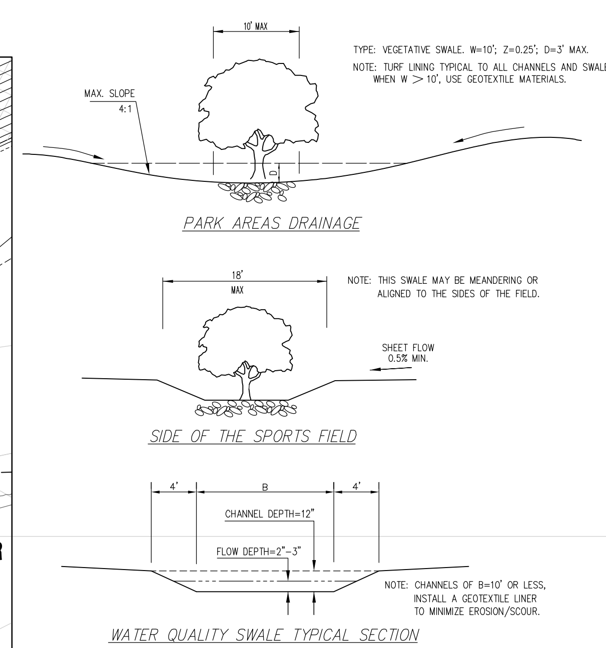
LANDMARK VILLAGE TRAIL SECTION (3)
NOT TO SCALE



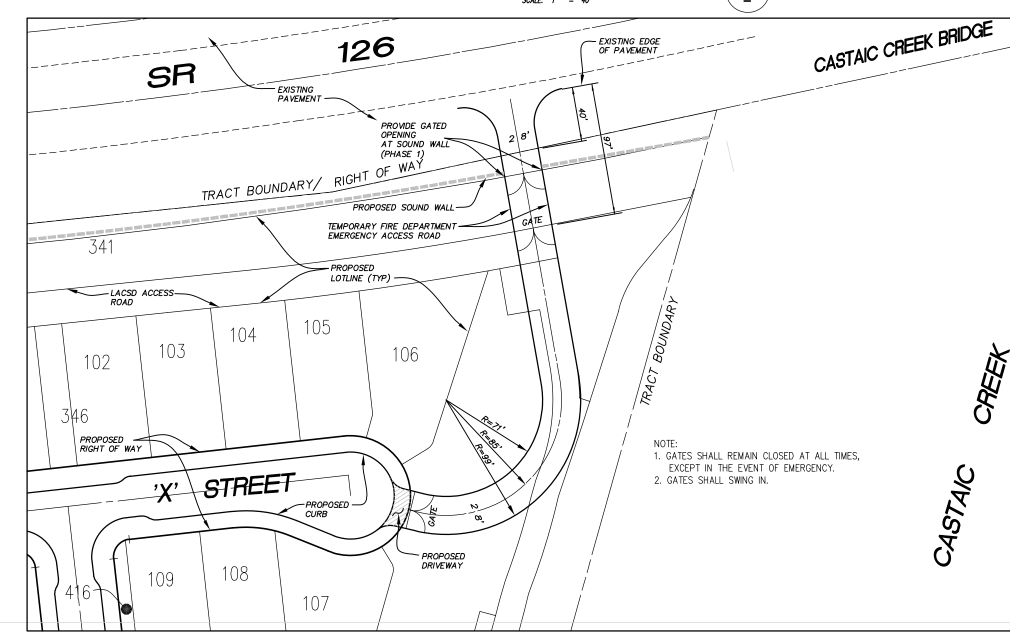
SHALLOW S.D. MANHOLE DETAIL (4)



ACCESS ROAD AT LONG CANYON BRIDGE (6)
1" = 40'



TYPICAL CONCEPT SECTION OF VEGETATIVE SWALE
NOT TO SCALE



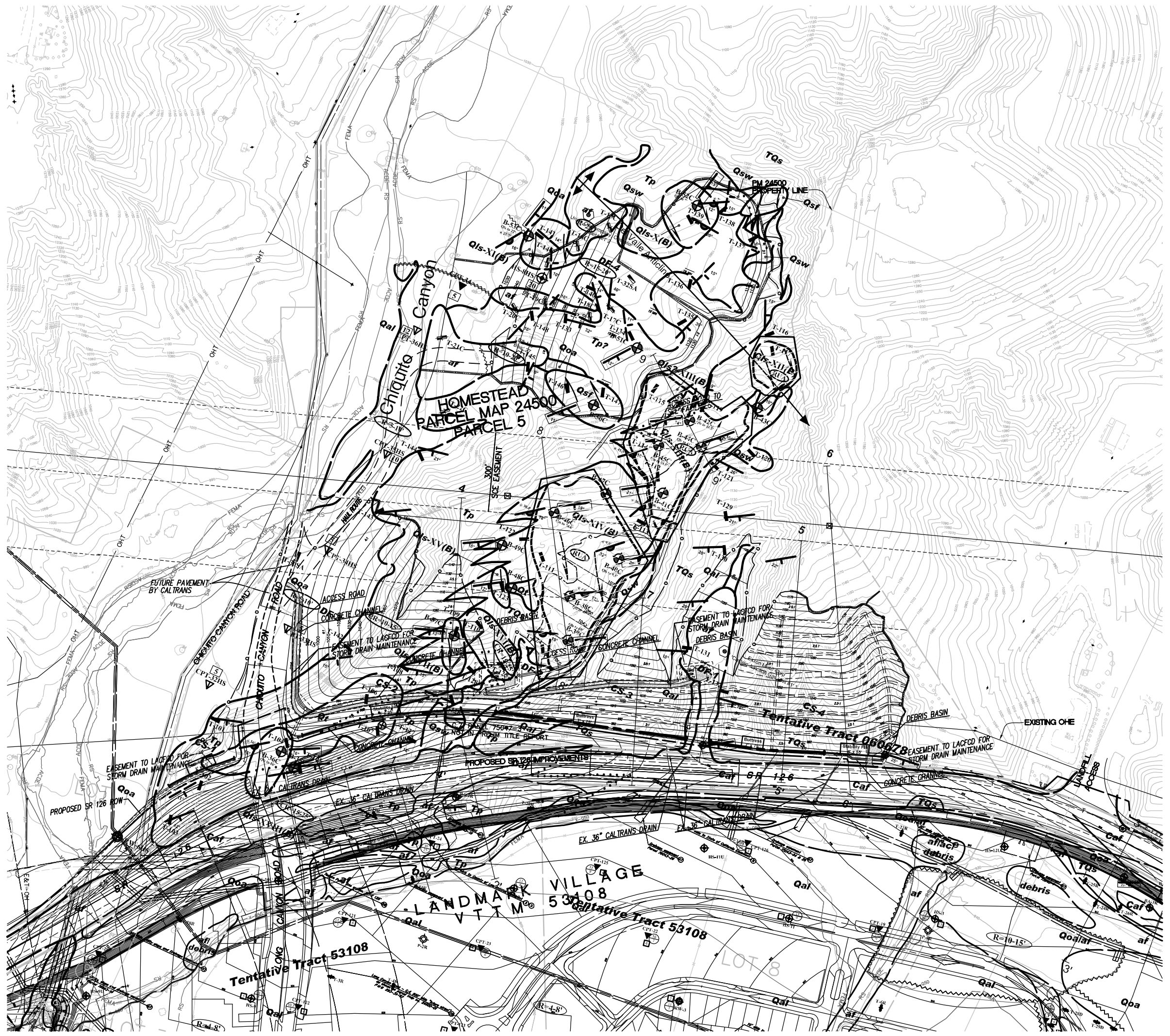
TEMPORARY FIRE DEPARTMENT EMERGENCY ACCESS ROAD (8)
SCALE: 1" = 40'

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION: PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.	PSOMAS — ERIK WINATA R.C.E. NO. C 67285 DATE	DESIGNED: NSK/YL; DRAFTED: TLC/JS; CHECKED: EW	OWNER/DEVELOPER: NEWHALL RANCH, 23823 WEST VALENCIA BOULEVARD, VALENCIA, CALIFORNIA 91355, TELEPHONE: (661) 255-4000, REPRESENTATIVE: MR. FRED MACMURDO	MAJOR LAND DIVISION VESTING TENTATIVE TRACT MAP NO. 53108 — DETAILS & SECTIONS — IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES, STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 12 OF 13

RV001-12.dwg



OFFSITE GRADING
CHIQUITO CANYON

CHIQUITO CANYON EARTHWORK VOLUMES	
RAW CUT	1,190,000 Cubic Yards
RAW FILL	1,071,000 Cubic Yards
NET CUT	1,190,000 Cubic Yards
NET FILL	1,190,000 Cubic Yards
EXPORT	0 Cubic Yards

LEGEND:

- EXISTING LOT LINE
- EXISTING EASEMENT
- 1100 PROPOSED CONTOUR
- DAYLIGHT LINE
- CUT / FILL — CUT & FILL
- FLOW LINE
- BENCH
- SWALE
- HAUL ROUTE ACCESS ROAD
- RC03 — RIVER CORRIDOR–2003 (FORMERLY SEA 23)
- RC — RIVER CORRIDOR (FORMERLY SEA 23)
- FEMA — FEMA
- Q50 — Q-CAP
- DEBRIS TRIBUTARY LIMIT
- SLOPE/ DEBRIS BASIN ACCESS
- DEBRIS BASIN
- OIL WELL GPS POSITIONED
- OIL WELL DEPT. OF OIL AND GAS CALCULATED POSITION
- OIL WELL DIGITIZED POSITION
- OIL WELL DEPT. OF OIL AND GAS METAL DETECTOR POSITION
- EXISTING OAK TREE
- RIP-RAP
- SPINE FLOWER
- SPINE FLOWER PROTECTION BOUNDARY

OFFSITE GRADING
ADOBE CANYON

ADOBE CANYON EARTHWORK VOLUMES	
RAW CUT	6,468,500 Cubic Yards
RAW FILL	133,000 Cubic Yards
NET CUT	6,468,500 Cubic Yards
NET FILL	146,300 Cubic Yards
EXPORT	6,322,200 Cubic Yards

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09	Job No.: 09-1702R-4
Scale: 1" = 100'	CAD File: Sheet 11 Revised Geologic Map
Sheet 11 — Geology by: BJS; Drawn by: GSD/JCH	Revised:

SEE 9/30/05 REPORT FOR VTT 060678
AND SHEET 3 FOR GEOLOGIC LEGEND

LEGAL DESCRIPTION:

PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28480 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
DRAFTED : TLC/JS
CHECKED : EW

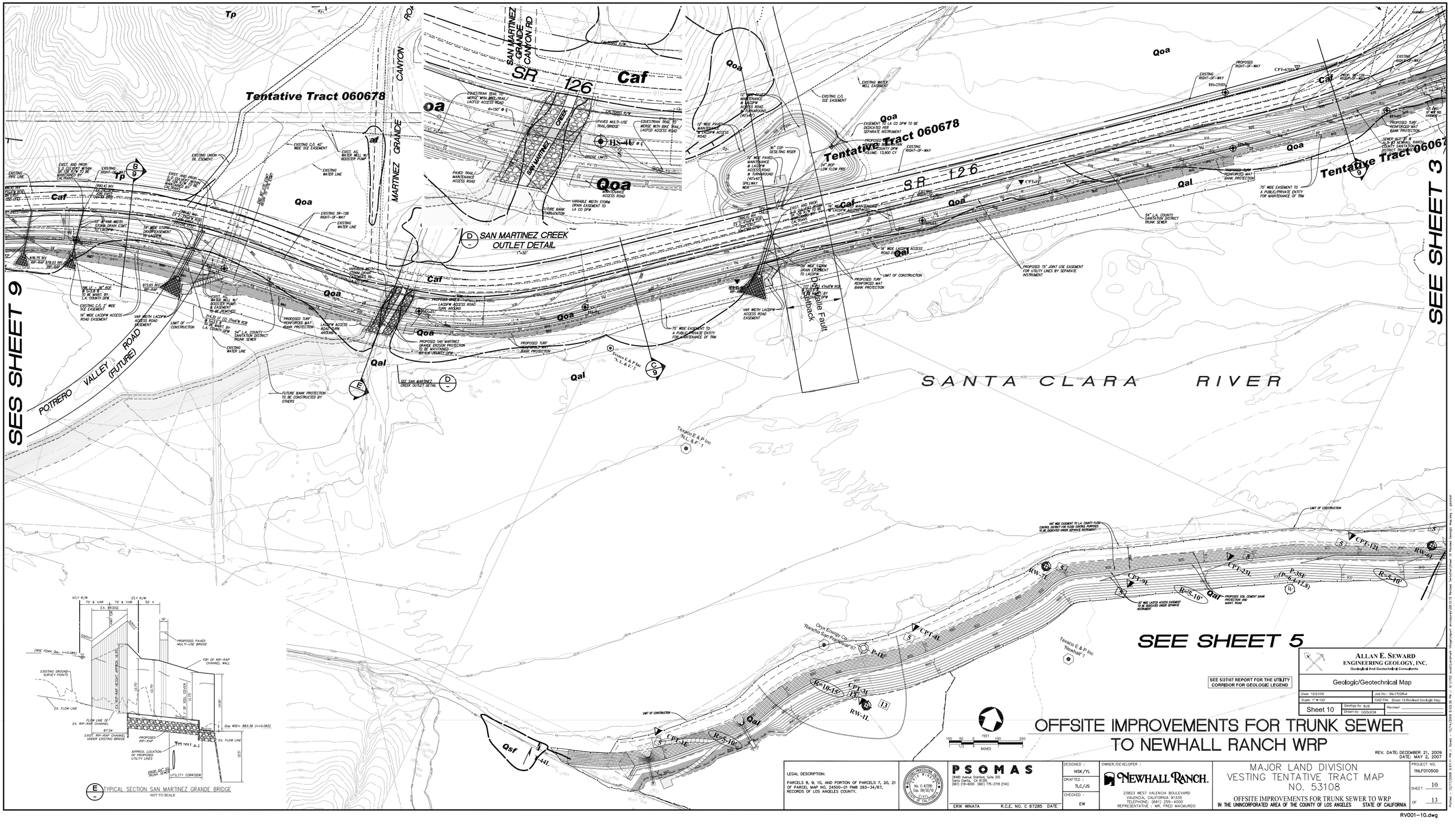
OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

CHIQUITO CANYON & ADOBE CANYON OFFSITE GRADING
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 11 OF 13

RV001-11.dwg



ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09 | Job No.: 09-170ZR-4
Scale: 1" = 100' | CAD File: Sheet 10 Revised Geologic Map
Sheet 10 | Geology by: BJS | Revised:
Drawn by: GSD/JCH

SEE 5/27/07 REPORT FOR THE UTILITY CORRIDOR FOR GEOLOGIC LEGEND

OFFSITE IMPROVEMENTS FOR TRUNK SEWER TO NEWHALL RANCH WRP

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER No. C-67285 (exp. 09/30/10) CIVIL STATE OF CALIFORNIA

PSOMAS
3648 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 259-4000 (661) 775-2778 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
DRAFTED : TLC/JS
CHECKED : EW

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

OFFSITE IMPROVEMENTS FOR TRUNK SEWER TO WRP
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 10 OF 13

RV001-10.dwg

WRP DEBRIS BASIN EARTHWORK VOLUMES	
RAW CUT	91,000 Cubic Yards
RAW FILL	2,000 Cubic Yards
NET CUT	91,000 Cubic Yards
NET FILL	2,200 Cubic Yards
EXPORT	88,800 Cubic Yards

UTILITY CORRIDOR GRADING EARTHWORK VOLUMES	
RAW CUT	3,200 Cubic Yards
RAW FILL	233,000 Cubic Yards
NET CUT	3,200 Cubic Yards
NET FILL	263,000 Cubic Yards
IMPORT	259,800 Cubic Yards

SEE SHEET 10

S R - 126

SANTA CLARA RIVER

Tentative Tract 060678

PROPOSED NEWHALL RANCH WASTE WATER RECLAMATION PLANT (N A P)

PROPOSED DEBRIS BASIN REQUIRED FOR WRP CONSTRUCTION VOLUME: 1,070 CY

PROPOSED DEBRIS BASIN REQUIRED FOR WRP CONSTRUCTION VOLUME: 1,080 CY

TEMPORARY CONSTRUCTION LIMIT LINE

EXISTING WATER LINE

EXISTING PACIFIC & TELEGRAPH EASEMENT

EASEMENT TO BE DEDICATED PER SEPARATE INSTRUMENT

EXISTING C/L 40' WIDE SCE EASEMENT

EXISTING RIGHT-OF-WAY

EXISTING SCE EASEMENT

EXIST. AG. WATER BOOSTER PUMP TO BE RELOCATED

ALT #1 PUMP STATION AT END OF SANITATION DISTRICT GRAVITY TRUNK SEWER (MAINTAINED BY NEWHALL RANCH COUNTY SANITATION DISTRICT) SEE SHEET 6

EXISTING PIPE LINE

12' WIDE ACCESS ROAD (LASPW)

UTIL. CONN. TO SLEQ678

10' & VAR. WIDTH STORM DRAIN EASEMENT

PROPOSED 90' EASEMENT FOR FUTURE UTILITY LINES SEE SHEET 6

ALT #1 PROPOSED FORCE MAIN TO EXISTING SEWER PUMP STATION (MAINTAINED BY NEWHALL RANCH COUNTY SANITATION DISTRICT) SEE SHEET 6

75' WIDE EASEMENT TO A PUBLIC/PRIVATE ENTITY FOR MAINTENANCE OF TRM

16' WIDE LASPW ACCESS ROAD EASEMENT

PROPOSED SOIL CEMENT BANK PROTECTION

186 LF - 36" RCP @ S=4.0% TO BE MAINT. BY LA. COUNTY DPW

150' WIDE EASEMENT TO LA CO SANITATION DISTRICT FOR MAINTENANCE OF SOIL CEMENT

JOIN EXISTING WRP IMPROVEMENTS

LIMITS OF CONSTRUCTION

END OF SOIL CEMENT BANK PROTECTION AND BEGIN MAT BANK PROTECTION

FUTURE BANK PROTECTION BY OTHERS

Approximate Location of Abandoned Southern Pacific Railroad Line

Approximate Location of Abandoned Route of State Highway 126 (Asphalt Still Present)

LOT 19

Qt Qaf Qoa Qal Tp

FEET 100 50 0 100 200

INCHES 1 1/2 0 1 2

OFFSITE IMPROVEMENTS FOR TRUNK SEWER TO NEWHALL RANCH WRP

TYPICAL SECTION A-A
BANK STABILIZATION - WRP
NOT TO SCALE

TYPICAL SECTION B-B
WRP TO POTRERO ROAD
NOT TO SCALE

TYPICAL SECTION C-C
POTRERO ROAD TO LANDMARK VILLAGE
NOT TO SCALE

* WO SWALE SIDE SLOPES ARE 4:1 MAX.

SEE 5/25/07 REPORT FOR THE UTILITY CORRIDOR FOR GEOLOGIC LEGEND

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/06 | Job No.: 09-1702R-4
Scale: 1" = 100' | CAD File: Sheet 9 Revised Geologic Map
Sheet 9 | Geology by: BJS | Revised:
Drawn by: GSD/JCH

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28480 Avenue Stanford, Suite 200
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(661) 259-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
DRAFTED : TLC/JS
CHECKED : EW

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

OFFSITE IMPROVEMENTS FOR TRUNK SEWER TO WRP
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 9 OF 13

RV001-09.dwg

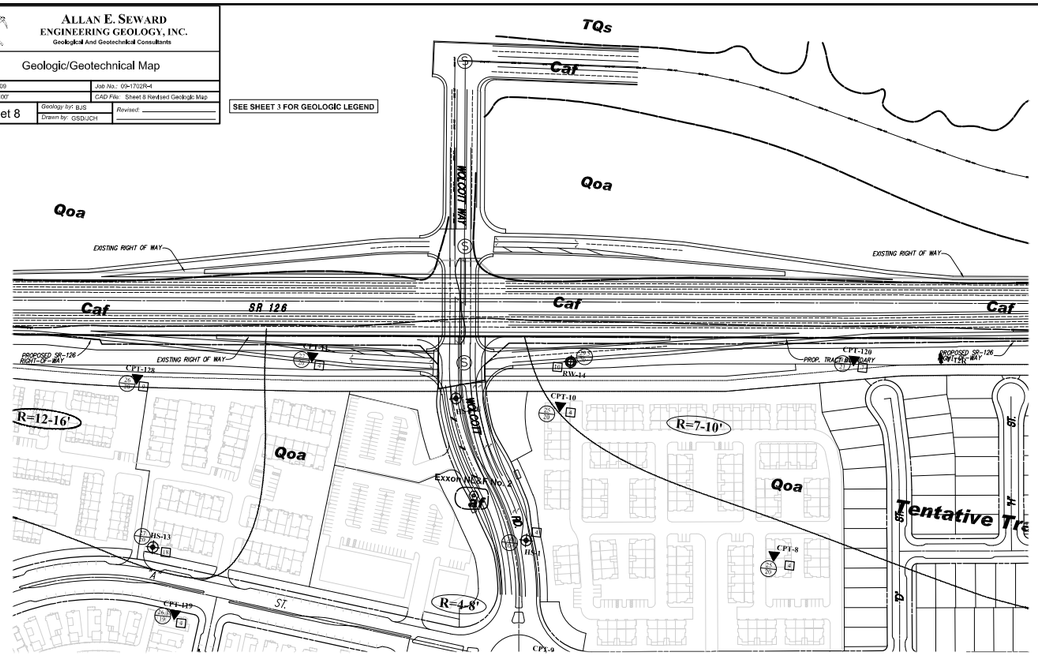
Model: --12/17/2009 3:48:27 PM-- Saved: 12/17/2009 3:48:27 PM -- Layout: 9 -- Plotted: 12/17/2009 3:49:31 PM -- By: GSD -- C:\Documents and Settings\Owner\Desktop\Current Working\Sheet 9 Revised Geologic Map.dwg -- plotted

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09 | Job No.: 09-170ZR-4
Scale: 1" = 100' | CAD File: Sheet 8 Revised Geologic Map
Sheet 8 | Geology by: BJS | Revised: ——
Drawn by: GSD/JCH

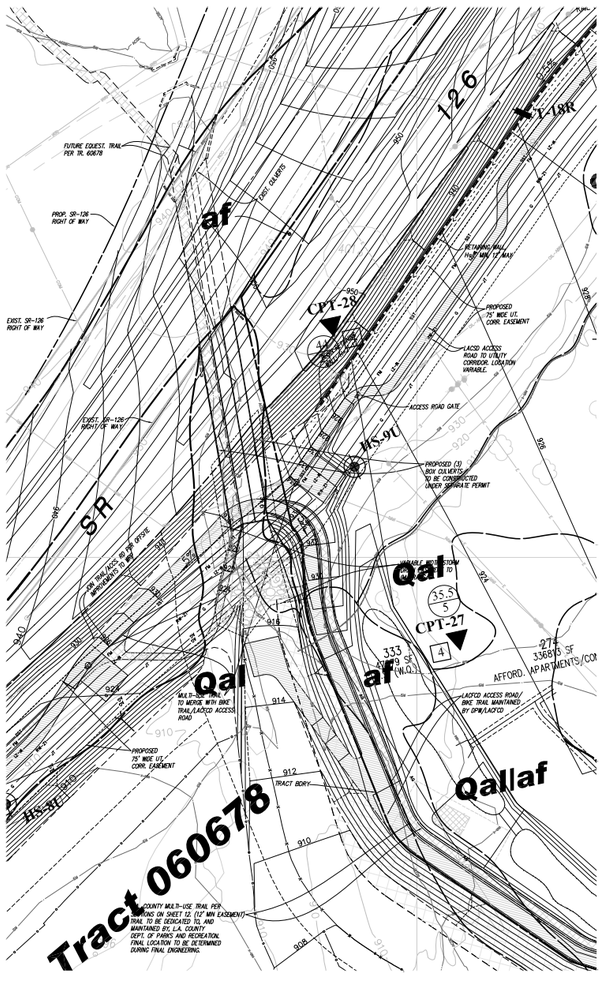
SEE SHEET 3 FOR GEOLOGIC LEGEND



POSSIBLE LAYOUT OF FUTURE INTERCHANGE WOLCOTT ROAD - SR 126

1 / –

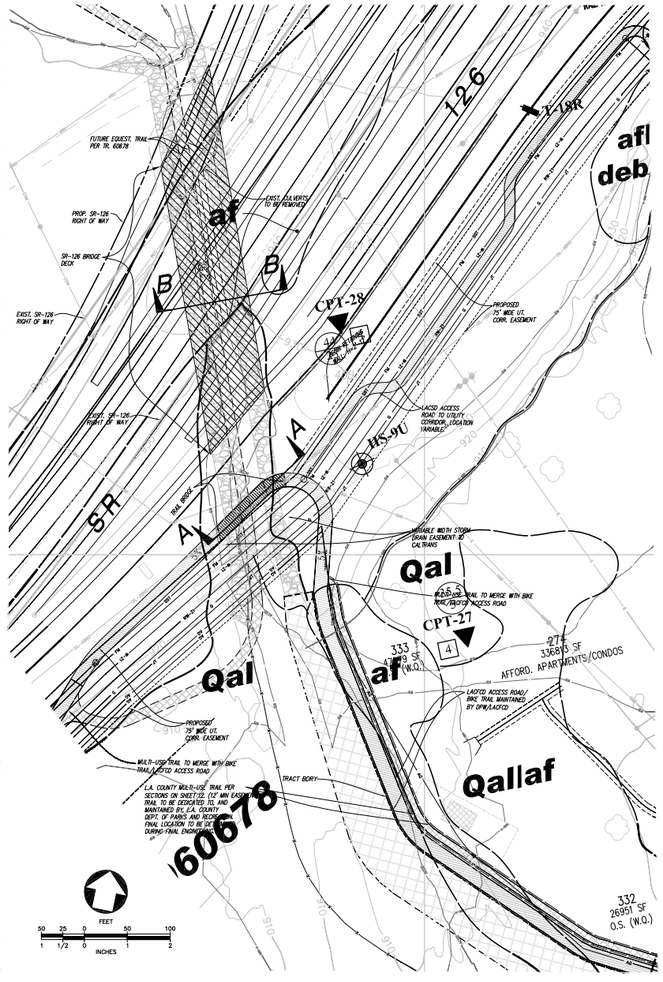
FEET 100 50 0 100 200
INCHES 1 1/2 0 1 2



CHIQUITO CYN OUTLET DETAIL 2

–

FEET 50 25 0 50 100
INCHES 1 1/2 0 1 2



CHIQUITO CYN OUTLET DETAIL BRIDGE ALTERNATE 3

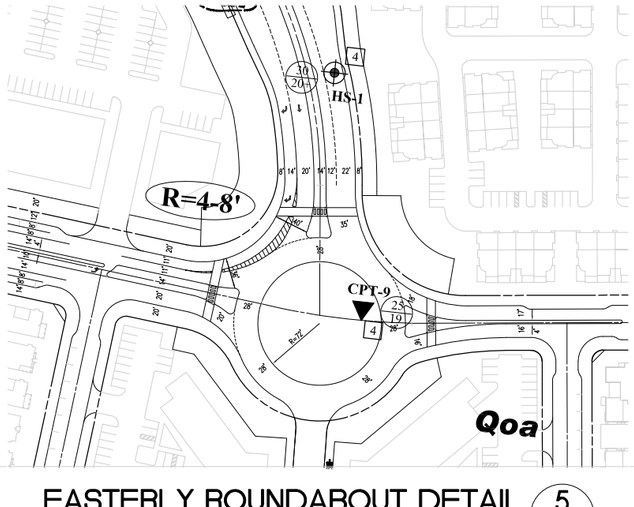
–

FEET 50 25 0 50 100
INCHES 1 1/2 0 1 2

WESTERLY ROUNDABOUT DETAIL 4

–

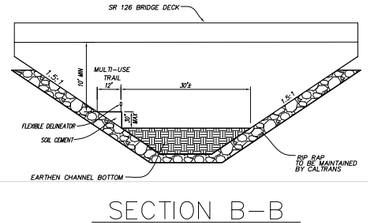
FEET 50 25 0 50 100
INCHES 1 1/2 0 1 2



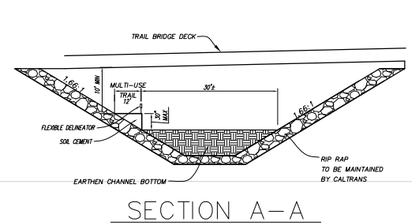
EASTERLY ROUNDABOUT DETAIL 5

–

FEET 50 25 0 50 100
INCHES 1 1/2 0 1 2



SECTION B–B
NOT TO SCALE



SECTION A–A
NOT TO SCALE

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
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REGISTERED PROFESSIONAL ENGINEER — ERIK WINATA — No. C-67285 — Exp. 09/30/10 — CIVIL — STATE OF CALIFORNIA

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28485 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 259-4000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
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CHECKED : EW

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

CHIQUITO LANDFILL DEBRIS BASIN & ALT. WOLCOTT/SR126 INTERCHANGE DETAILS
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 8 OF 13

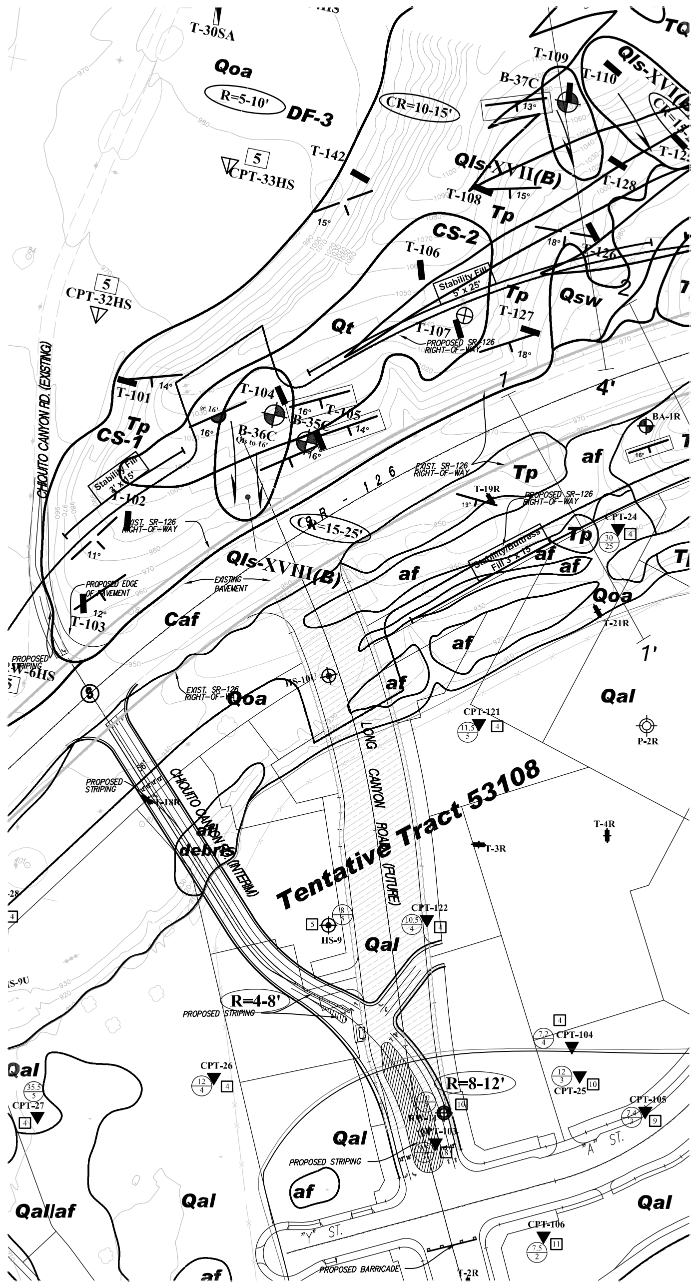
RV001–08.dwg

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

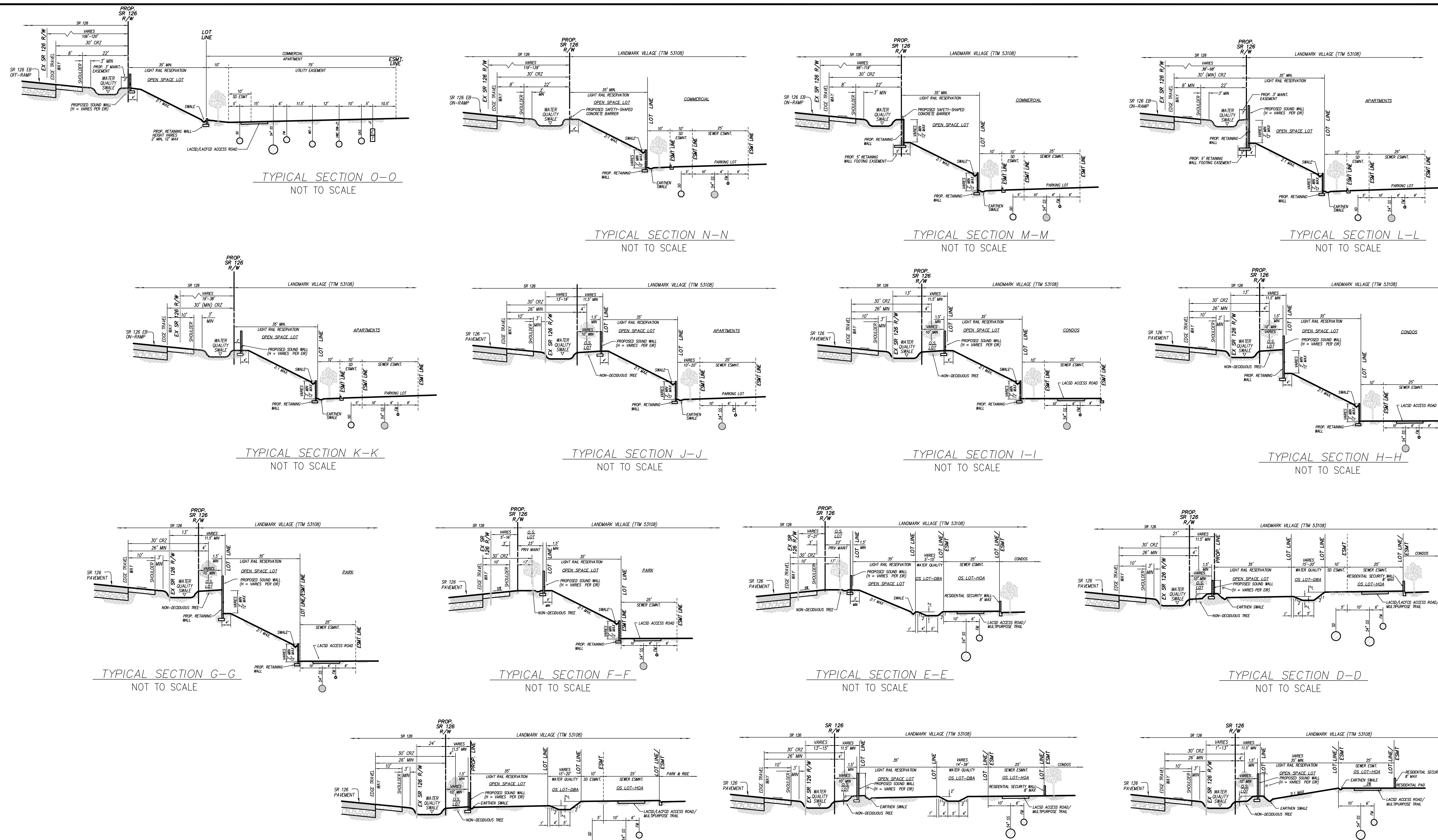
Geologic/Geotechnical Map

Date: 12/01/09 | Job No.: 09-1702R-4
Scale: 1" = 100' | CAD File: Sheet 7 Revised Geologic Map
Sheet 7 | Geology by: BJS | Revised:
Drawn by: GSD/JCH

SEE SHEET 3 FOR GEOLOGIC LEGEND



INTERIM DESIGN CHIQUITO CANYON RD. - SR 126 INTERCHANGE



TYPICAL SECTION O-O
NOT TO SCALE

TYPICAL SECTION N-N
NOT TO SCALE

TYPICAL SECTION M-M
NOT TO SCALE

TYPICAL SECTION L-L
NOT TO SCALE

TYPICAL SECTION K-K
NOT TO SCALE

TYPICAL SECTION J-J
NOT TO SCALE

TYPICAL SECTION I-I
NOT TO SCALE

TYPICAL SECTION H-H
NOT TO SCALE

TYPICAL SECTION G-G
NOT TO SCALE

TYPICAL SECTION F-F
NOT TO SCALE

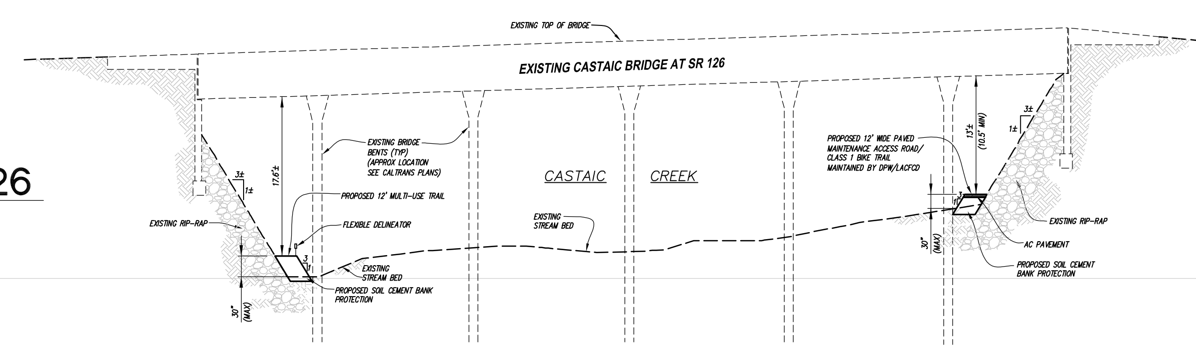
TYPICAL SECTION E-E
NOT TO SCALE

TYPICAL SECTION D-D
NOT TO SCALE

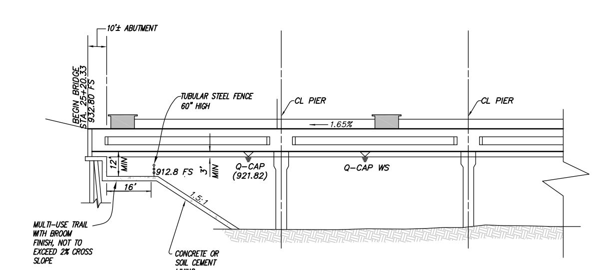
TYPICAL SECTION C-C
NOT TO SCALE

TYPICAL SECTION B-B
NOT TO SCALE

TYPICAL SECTION A-A
NOT TO SCALE



TRAIL SECTION AT CASTAIC CREEK BRIDGE
NOT TO SCALE



TRAIL SECTION AT LONG CANYON BRIDGE
NOT TO SCALE

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

PSOMAS
ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED: NSK/YL
DRAFTED: TLC/JS
CHECKED: EW

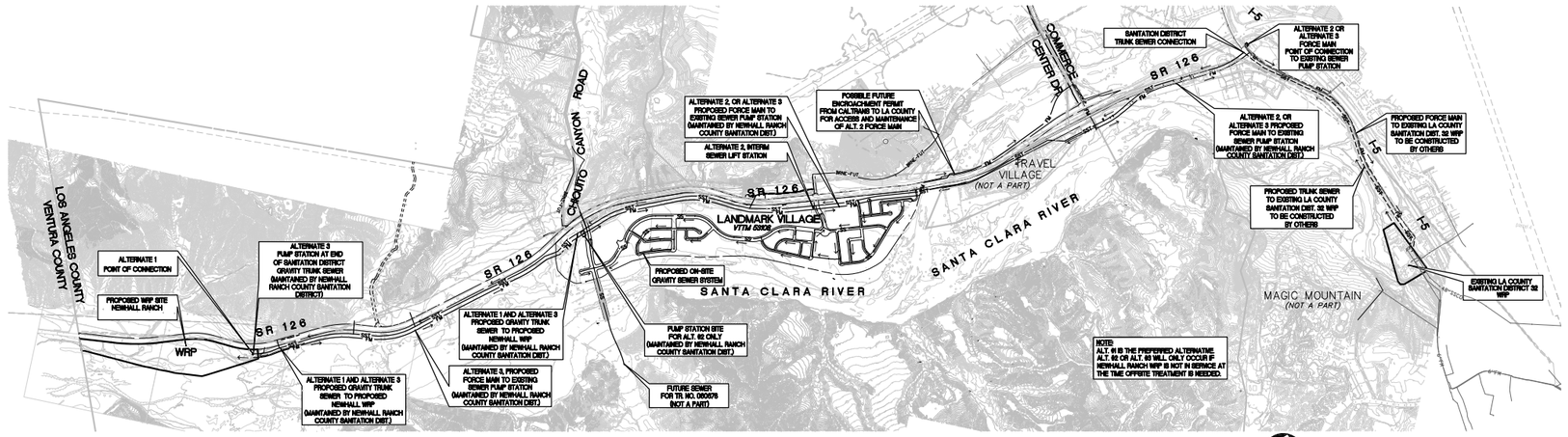
OWNER/DEVELOPER:
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91355
TELEPHONE: (661) 255-4000
REPRESENTATIVE: MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

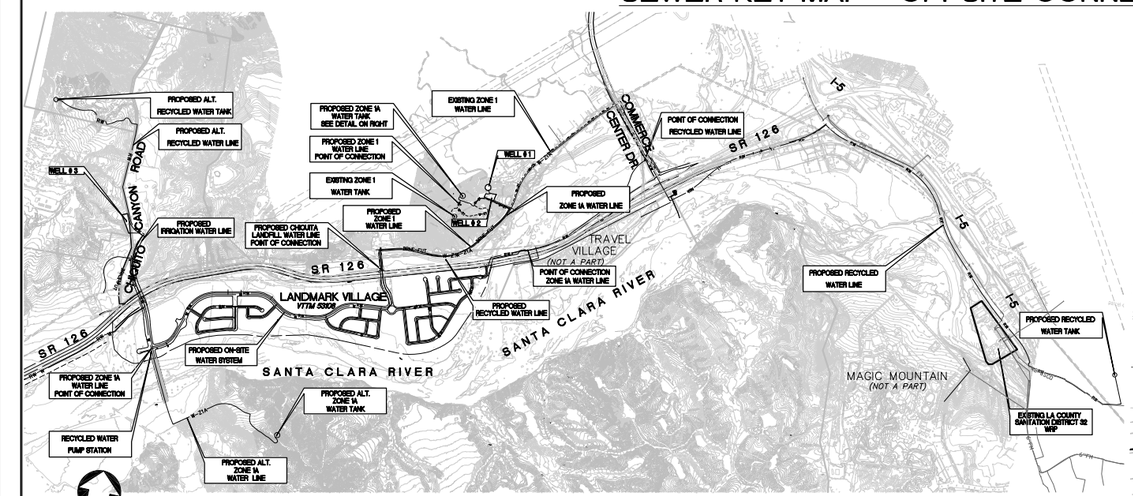
INTERIM DESIGN LONG CANYON - SR 126; UTIL. CORR. SECTIONS
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 7 OF 13

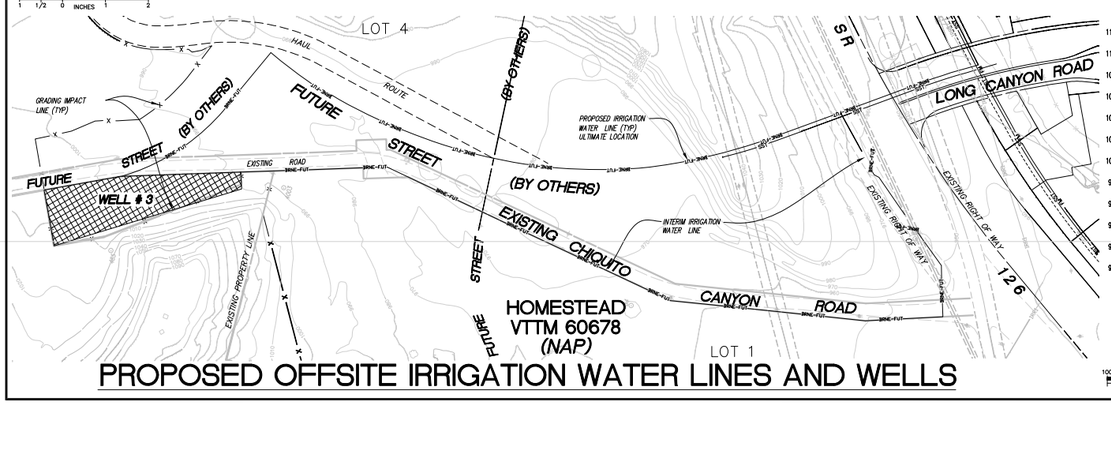
RV001-07.dwg



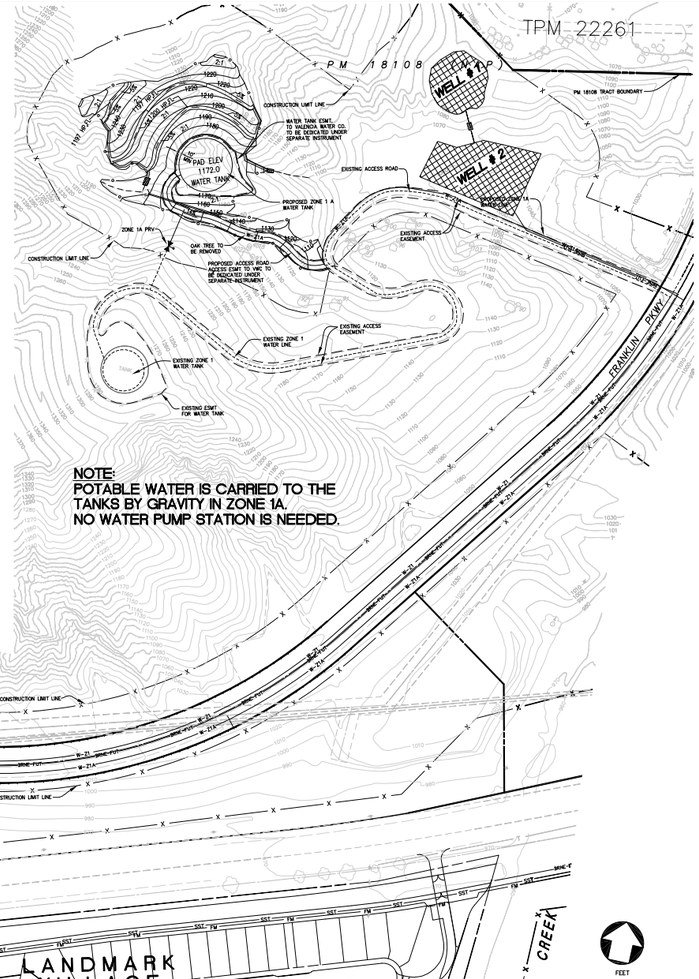
SEWER KEY MAP - OFFSITE CONNECTION



WATER KEY MAP - OFFSITE CONNECTION

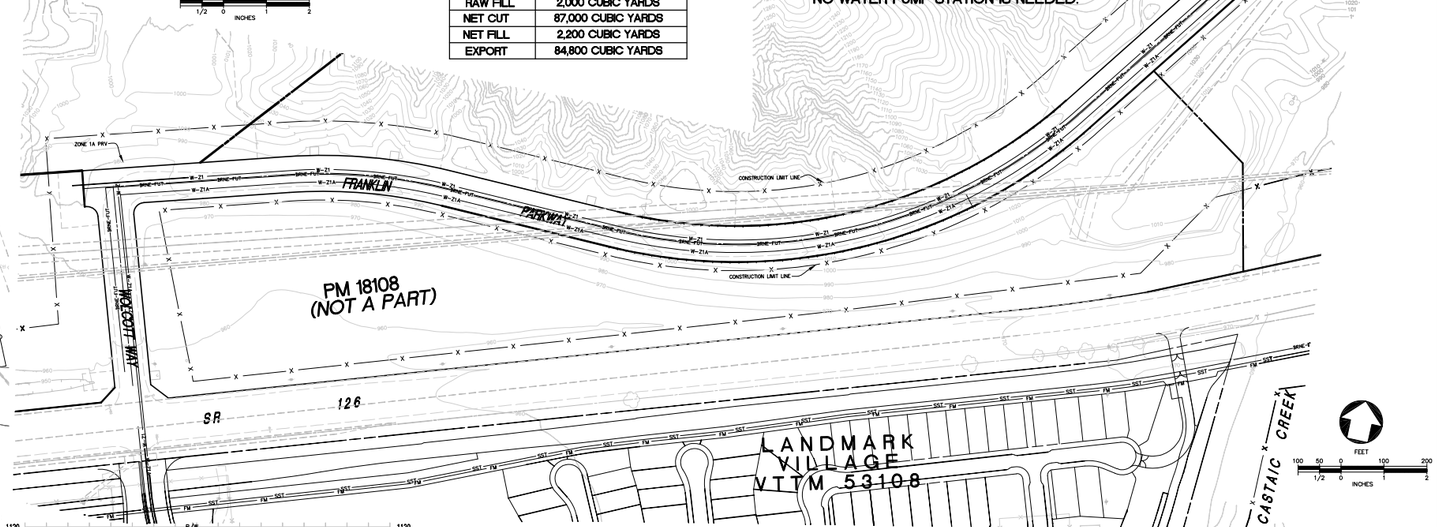


PROPOSED OFFSITE IRRIGATION WATER LINES AND WELLS

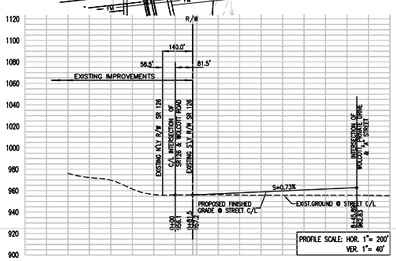


NOTE:
POTABLE WATER IS CARRIED TO THE TANKS BY GRAVITY IN ZONE 1A. NO WATER PUMP STATION IS NEEDED.

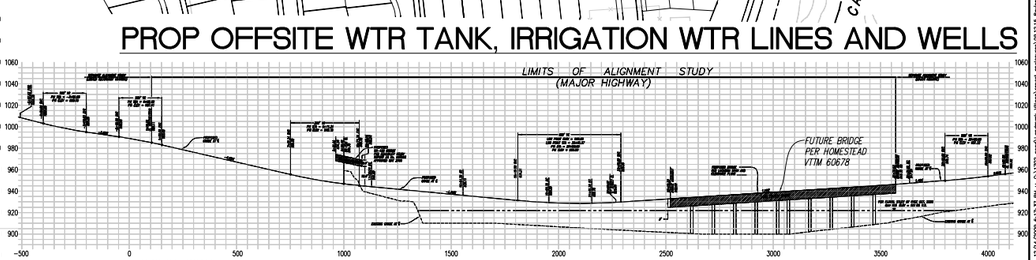
OFFSITE ZONE 1A WATER TANK SITE	
RAW CUT	87,000 CUBIC YARDS
RAW FILL	2,000 CUBIC YARDS
NET CUT	87,000 CUBIC YARDS
NET FILL	2,200 CUBIC YARDS
EXPORT	84,800 CUBIC YARDS



PROP OFFSITE WTR TANK, IRRIGATION WTR LINES AND WELLS



WOLCOTT ROAD PROFILE



LONG CANYON ROAD PROFILE

PROFILE SCALE:
HORIZ: 1"=200'
VERT: 1"=40'

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
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REGISTERED PROFESSIONAL ENGINEER — No. C-67285 — Exp. 09/30/10 — CIVIL — STATE OF CALIFORNIA

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(661) 259-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED :	NSK/YL
DRAFTED :	TLC/JS
CHECKED :	EW

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108
SEWER & WATER KEY MAPS, LONG CANYON RD & WOLCOTT RD PROFILES
PROPOSED OFFSITE IRRIGATION WATER LINES AND WELLS
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 6 OF 13

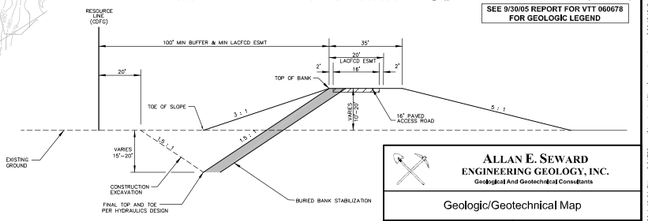
RV001-06.dwg

SEE SHEET 3

SOUTH BANK TEMP. LEVEE EARTHWORK VOLUME	
RAW CUT	0 Cubic Yards
RAW FILL	250,000 Cubic Yards
NET CUT	0 Cubic Yards
NET FILL	316,000 Cubic Yards
IMPORT	316,000 Cubic Yards

SEE SHEET 4

SEE 9/30/05 REPORT FOR VTT 060678 FOR GEOLOGIC LEGEND



A BANK STABILIZATION TYPICAL SECTION
NTS

100' 50' 0' 100' 200'
GRAPHIC SCALE
Note: For reduced sized prints, original scale is in inches

ALLAN E. SEWARD ENGINEERING GEOLOGY, INC.
Geologicid And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09	Job No.: 09-170ZR-4
Scale: 1" = 100'	CAD File: Sheet 5 Revised Geologic Map
Sheet 5	Geology by: BJS / Drawn by: GSD/JCH / Assisted

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:

PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

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2940 Ahrens Stanton, Suite 200
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(661) 259-6000 (661) 775-2778 (FAX)
ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
DRAFTED : TLC/JS
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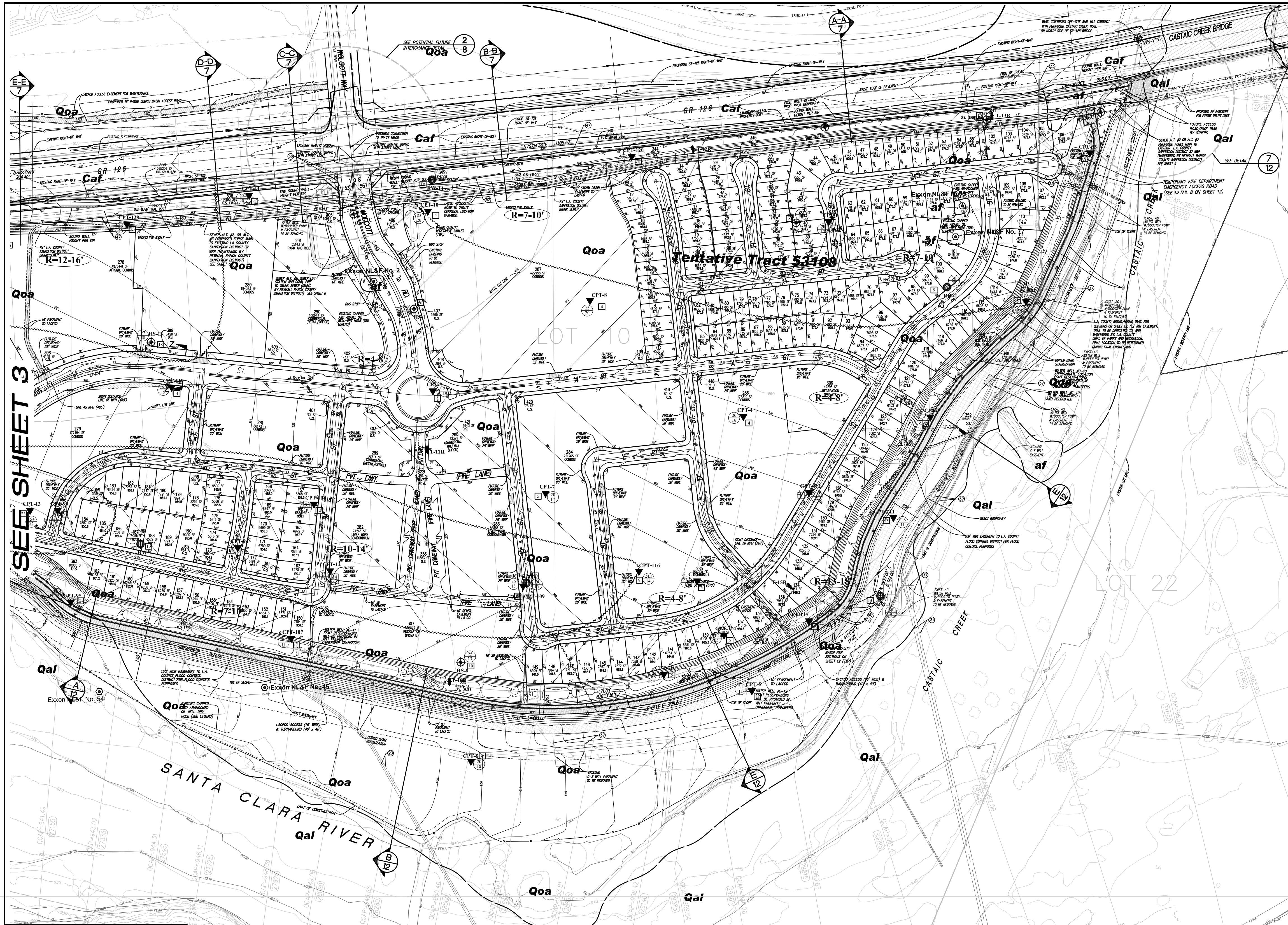
OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91355
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

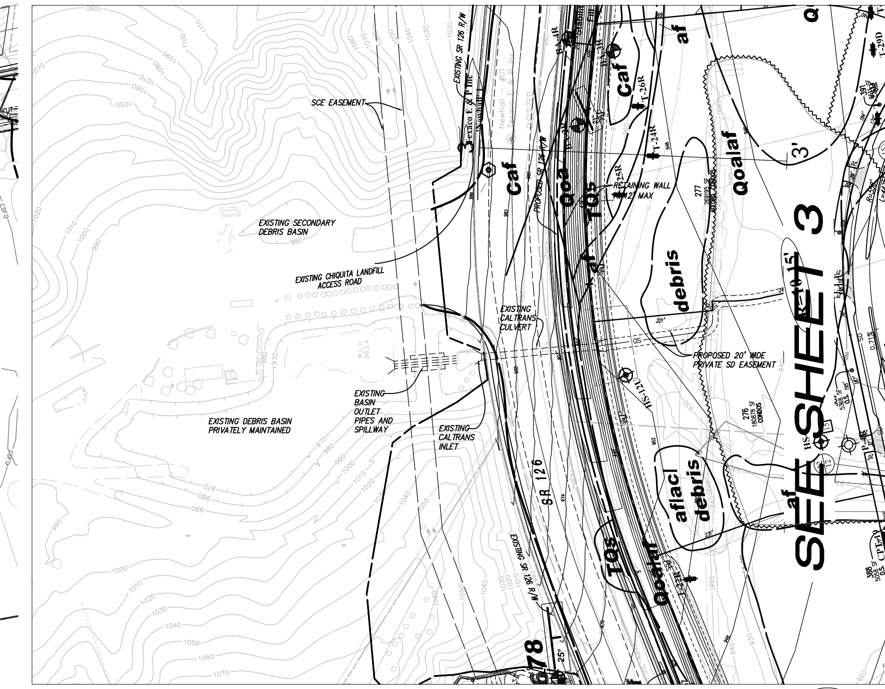
MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 5 OF 13

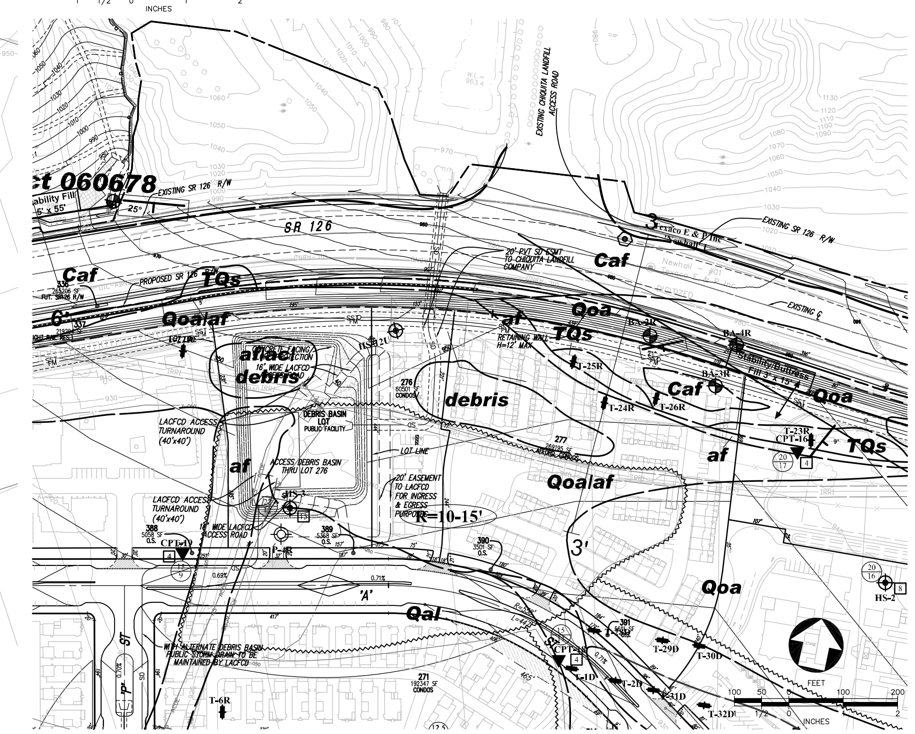
RV001-05.dwg





EXISTING LANDFILL DEBRIS BASIN AS SHOWN ON VTTM

5
—



ALTERNATE DEBRIS BASIN LOCATION

6
—

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

ALLAN E. SEWARD ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09	Job No.: 09-1702R-4	
Scale: 1" = 100'	CAD File: Sheet 4 Revised Geologic Map	
Sheet 4	Geology by: BJS / Drawn by: GSD/JCH	Revised:

SEE SHEET 3 FOR GEOLOGIC LEGEND

100' 50' 0' 100' 200'
GRAPHIC SCALE
Note: For reduced sized prints, original scale is in inches

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REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

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(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

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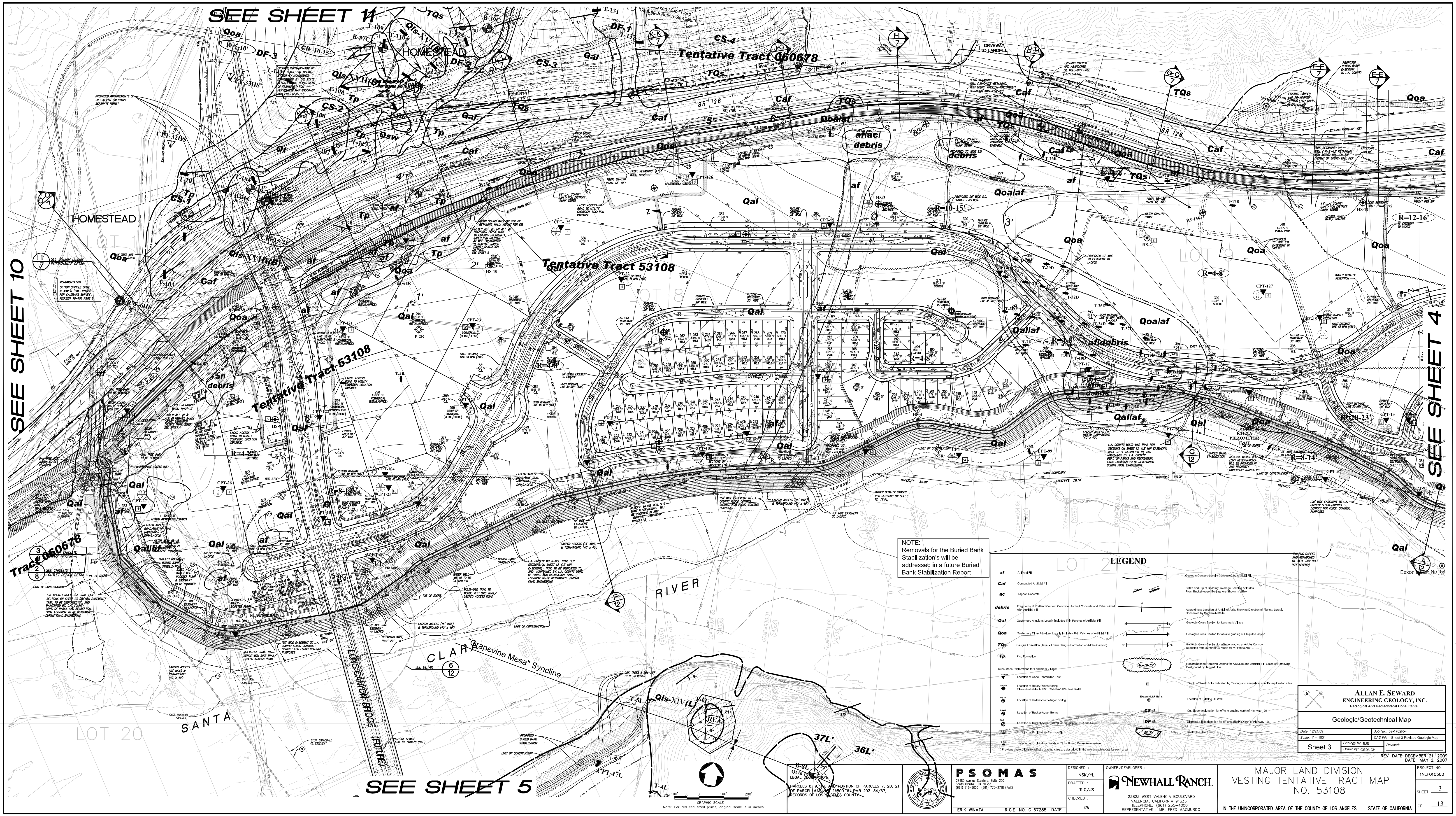
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION VESTING TENTATIVE TRACT MAP NO. 53108

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 4 OF 13

RV001-04.dwg


SEE SHEET 11
SEE SHEET 10
SEE SHEET 4
SEE SHEET 5
Tentative Tract 060678
Tentative Tract 53108
HOMESTEAD
SR 126
RIVER
CLARA
SANTA
"Grapevine Mesa" Syncline
LONG CANYON BRIDGE (FUTURE)
LOT 20
LOT 21
NOTE:
Removals for the Buried Bank Stabilization's will be addressed in a future Buried Bank Stabilization Report
LEGEND
af Artificial Fill
Caf Compacted Artificial Fill
ac Asphalt Concrete
debris Fragments of Portland Cement Concrete, Asphalt Concrete and Rebar mixed with Artificial Fill
Qal Quaternary Alluvium; Locally Includes Thin Patches of Artificial Fill
Qoa Quaternary Older Alluvium; Locally Includes Thin Patches of Artificial Fill
TQs Saugus Formation (TQsL = Lower Saugus Formation at Adobe Canyon)
Tp Pico Formation
Subsurface Explorations for Landmark Village*
Location of Cone Penetration Test
Location of Rotary-Wash Boring (Piezometers Installed In RW-1, RW-3, RW-7, RW-11 and RW-13)
Location of Hollow-Stem-Auger Boring
Location of Bucket-Auger Boring
Location of Bucket-Auger Boring for cut-slopes CS-3 and CS-4
Location of Exploratory Backhoe Pit
Location of Exploratory Backhoe Pit for Buried Debris Assessment
* Previous explorations for off-site grading sites are described in the referenced reports for each area
Geologic Contact, Locally Concealed by Artificial Fill
Strike and Dip of Bedding; Average Bedding Attitudes From Bucket-Auger Borings Are Shown In a Box
Approximate Location of Anticline Axis; Showing Direction of Plunge; Largely Concealed by Surficial Material
Geologic Cross Section for Landmark Village
Geologic Cross Section for off-site grading at Chiquito Canyon
Geologic Cross Section for off-site grading at Adobe Canyon (modified from our 9/30/05 report for VTT 060678)
Recommended Removal Depths for Alluvium and Artificial Fill; Limits of Removals Designated by Jagged Line
Depth of Weak Soils Indicated by Testing and analysis at specific exploration sites
Location of Existing Oil Well
Cut Slope designation for off-site grading north of Highway 126
Disposal Fill designation for off-site grading north of Highway 126
Restricted Use Area
ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants
Geologic/Geotechnical Map
Date: 12/21/09
Job No.: 09-1702R-4
Scale: 1" = 100'
CAD File: Sheet 3 Revised Geologic Map
Sheet 3
Geology by: BJS
Drawn by: GSD/JCH
Revised:
REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007
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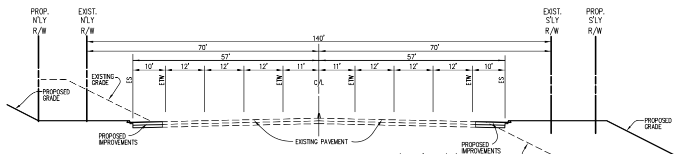
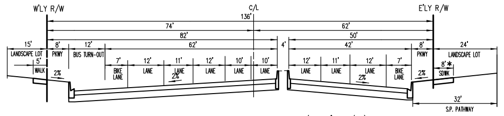
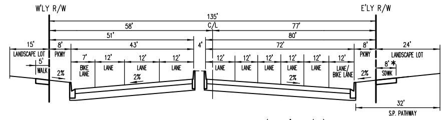
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA
PROJECT NO. 1NLF010500
SHEET 3 OF 13
RV001-03.dwg



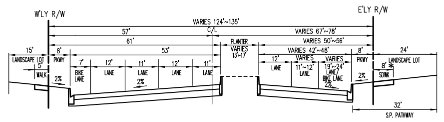
TYPICAL SR 126 STREET SECTION (140' R/W)



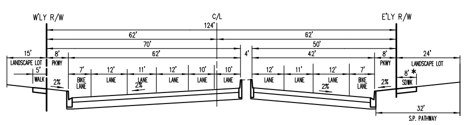
TYPICAL STREET SECTION (136' R/W)
LONG CANYON ROAD (North Side of 'A' Street)
Major Highway, Case 1, Section C
With Bus Turn-Out, On Street Bike Lane
No On Street Parking
60 MPH Design Speed



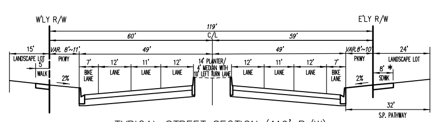
TYPICAL STREET SECTION (135' R/W)
LONG CANYON ROAD (South Side of SR 126)
Major Highway, Case 1, Section C
With On Street Bike Lane
No On Street Parking
60 MPH Design Speed



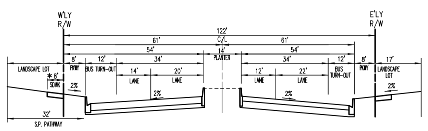
TYPICAL STREET SECTION (124'–135' R/W)
LONG CANYON ROAD (Betwen SR 126 & 'A' Street)
Major Highway, Case 1, Section C
With On Street Bike Lane
No On Street Parking
60 MPH Design Speed



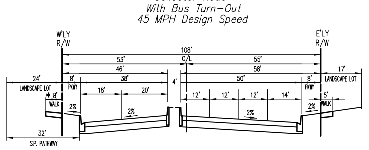
TYPICAL STREET SECTION (124' R/W)
LONG CANYON ROAD (North Side of 'A' Street)
Major Highway, Case 1, Section C
With On Street Bike Lane
No On Street Parking
60 MPH Design Speed



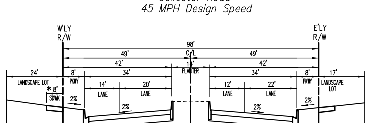
TYPICAL STREET SECTION (119' R/W)
LONG CANYON ROAD (South Side of 'A' STREET)
Major Highway, Case 1, Section C
With On Street Bike Lane
No On Street Parking
65 MPH Design Speed



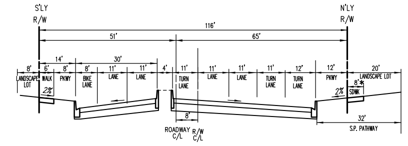
TYPICAL STREET SECTION (98' R/W)
WOLCOTT ROAD (North Side of 'A' STREET)
Collector Road
With Bus Turn-Out
45 MPH Design Speed



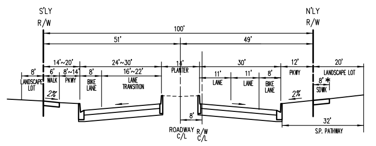
TYPICAL STREET SECTION (108' R/W)
WOLCOTT ROAD (South Side of SR 126)
Collector Road
45 MPH Design Speed



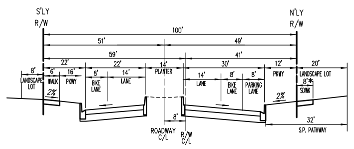
TYPICAL STREET SECTION (98' R/W)
WOLCOTT ROAD (North Side of 'A' STREET)
Collector Road
45 MPH Design Speed



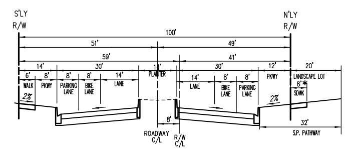
TYPICAL STREET SECTION (116' R/W)
"A" STREET (Spine Road Collector)
From Long Canyon Road to 200' East of Long Canyon Road
Local Access – 45 MPH Design Speed
With On Street Bike Lane
No On Street Parking



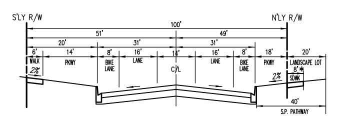
TYPICAL STREET SECTION (100' R/W)
"A" STREET (Spine Road Collector)
From 200' East of Long Canyon Road to Westerly Round About
Local Access – 45 MPH Design Speed
With On Street Bike Lane
No On Street Parking



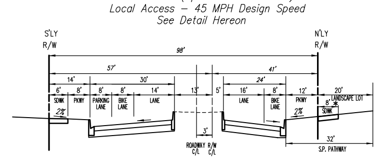
TYPICAL STREET SECTION (100' R/W)
"A" STREET (Spine Road Collector)
From Westerly Round About to "N" Street and
Local Access – 45 MPH Design Speed
With On Street Parking and On Street Bike Lane



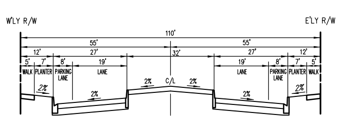
TYPICAL STREET SECTION (100' R/W)
"A" STREET (Spine Road Collector)
From "N" Street to "T" Street
From "J" Street to Wolcott Rd.
Local Access – 45 MPH Design Speed
With On Street Parking and On Street Bike Lane



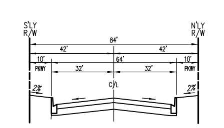
TYPICAL STREET SECTION Z – Z
"A" STREET (Spine Road Collector)
Local Access – 45 MPH Design Speed
See Detail Hereon



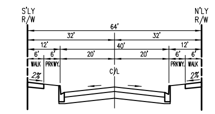
TYPICAL SECTION (98' R/W)
"A" STREET (Spine Road Collector)
From "T" Street TO "J" Street
Local Access – 45 MPH Design Speed
With On Street Parking Lane (South Side) and On Street Bike Lane



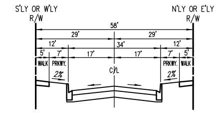
TYPICAL SECTION (110' R/W)
"P" DRIVE
Local Access – 30 MPH Design Speed



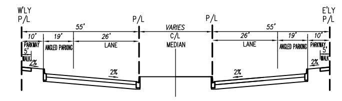
TYPICAL LOCAL STREET SECTION (84' R/W)
"Y" STREET (Industrial Collector)
Local Access – 30 MPH Design Speed



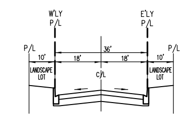
TYPICAL LOCAL STREET SECTION (64' R/W)
"A" STREET (From Wolcott Road to "D" Street)
Local Access – 30 MPH Design Speed



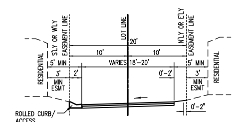
TYPICAL LOCAL STREET SECTION (58' R/W)
"A" STREET (FROM "D" STREET TO "C" STREET), "B", "C, "D" ,"E", "F" ,"H", "I", "J", "K", "L"
, M", "N", "O", "Q", "R", "S", "T", "W", "X", "Z" STREETS
Local Access – 30 MPH Design Speed



TYPICAL CONCEPT SECTION
Private Driveway South of "A" Street
at Wolcott Road
Local Access – 30 MPH Design Speed



TYPICAL CONCEPT SECTION
Private Driveway South of "A" Street
at Wolcott Road
With No Median
Local Access – 30 MPH Design Speed

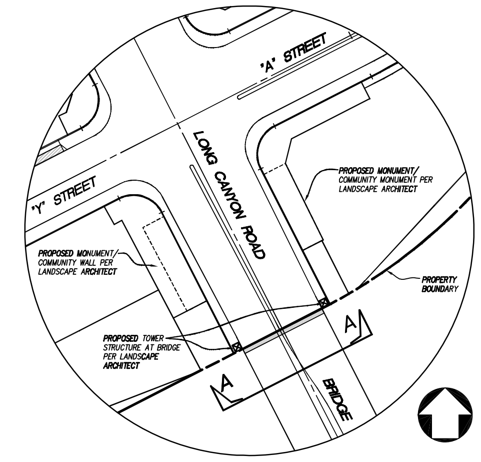


TYPICAL PRIVATE DRIVEWAY (20' EASEMENT)

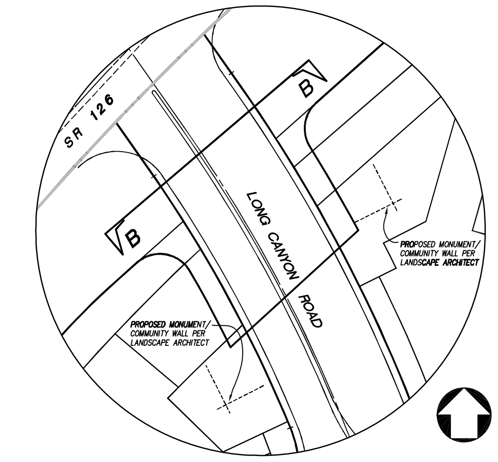
NOTE:
** 8' SIDEWALK WILL BE MEANDERING ENTIRELY WITHIN THE 24' OR 20' WIDE LANDSCAPE LOT*

STREET TYPICAL SECTIONS

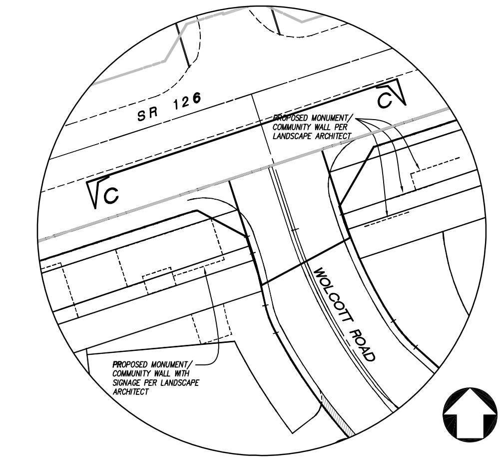
NOT TO SCALE



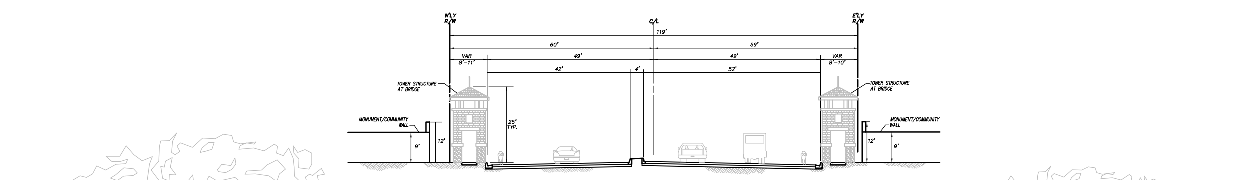
ENTRY AT LONG CANYON BRIDGE
SCALE: 1" = 60'



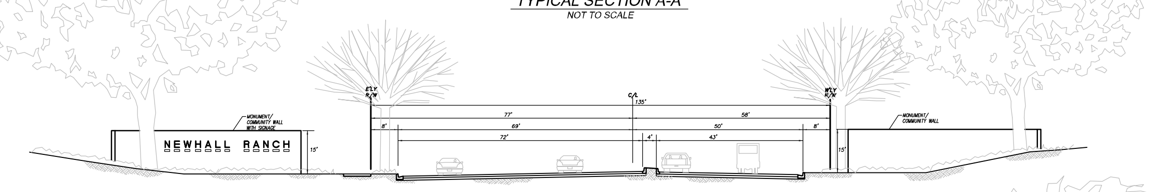
ENTRY AT LONG CANYON - SOUTH SR 126
SCALE: 1" = 60'



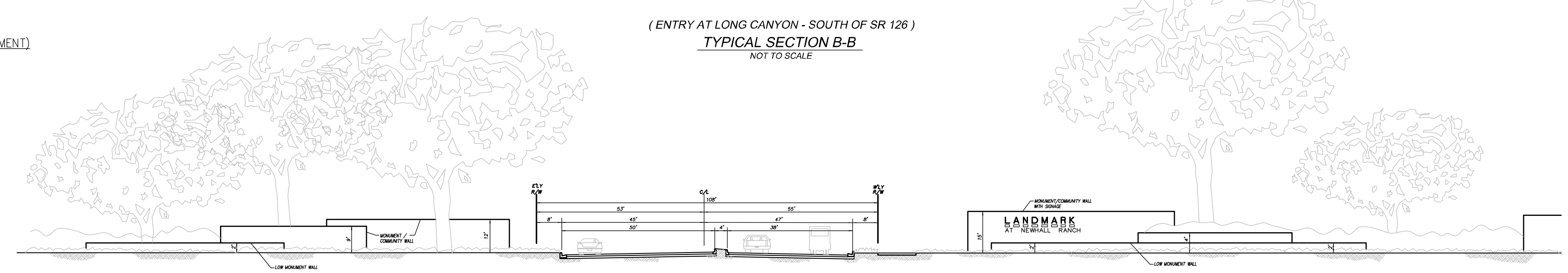
LANDMARK VILLAGE ENTRY AT WOLCOTT ROAD
SCALE: 1" = 60'



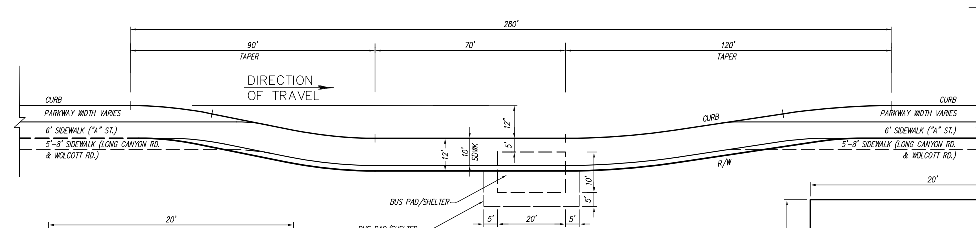
(ENTRY AT LONG CANYON BRIDGE)
TYPICAL SECTION A-A
NOT TO SCALE



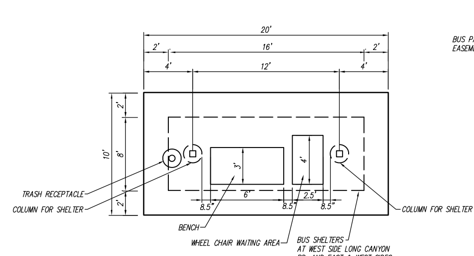
(ENTRY AT LONG CANYON - SOUTH OF SR 126)
TYPICAL SECTION B-B
NOT TO SCALE



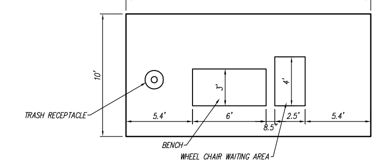
(LANDMARK VILLAGE ENTRY AT WOLCOTT ROAD SOUTH OF SR-126)
TYPICAL SECTION C-C
NOT TO SCALE



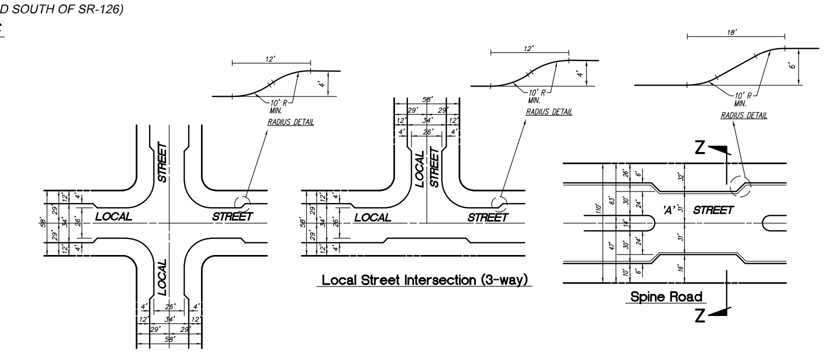
BUS TURN-OUT DETAIL
NOT TO SCALE



BUS PAD W/ SHELTER DETAIL
NOT TO SCALE



BUS PAD W/O SHELTER DETAIL
NOT TO SCALE



SPINE ROAD AND LOCAL STREET INTERSECTION DETAIL
NOT TO SCALE

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION		ENGINEER		OWNER/DEVELOPER		
LEGAL DESCRIPTION: PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.	REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA	PSOMAS 28480 Avenue Stanford, Suite 200 Santa Clarita, CA 91355 (661) 219-6000 (661) 775-2718 (FAX) ERIK WINATA R.C.E. NO. C 67285 DATE	DESIGNED : NSK/YL DRAFTED : TLC/JS CHECKED : EW	OWNER/DEVELOPER : NEWHALL RANCH. 23823 WEST VALENCIA BOULEVARD VALENCIA, CALIFORNIA 91335 TELEPHONE: (661) 255-4000 REPRESENTATIVE : MR. FRED MACMURDO	MAJOR LAND DIVISION VESTING TENTATIVE TRACT MAP NO. 53108 IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA	PROJECT NO. 1NLF010500 SHEET 2 OF 13

RV001–02.dwg

Plotted – 12/17/2009 3:15:58 PM :: Saved – 12/15/2009 2:17:45 PM

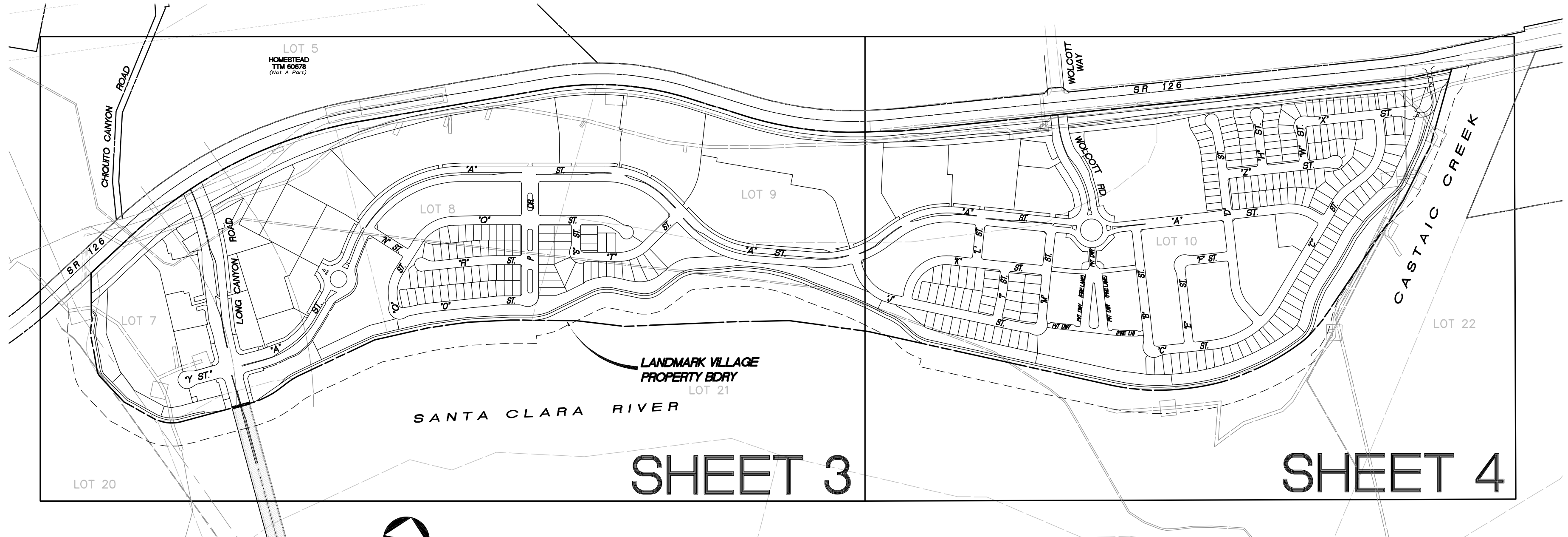
LOT SUMMARY

LOT NUMBERS	TOTAL LOTS/ BLDGS	TYPE (USE)	LOT SIZE	TOTAL D.U.	TOTAL S.F.D. or DET. CONDOS D.U.	MF D.U.s in bldgs w/1-4 D.U.s/bldg	MF D.U.s in bldgs w/5+ D.U.s/bldg
1–93	93/93	S.F.D 45' x 100' 4,500 S.F.	472,574 S.F.	93	93	0	0
94–190	97/97	S.F.D 55' x 100' 5,500 S.F.	663,229 S.F.	97	97	0	0
191–270	80/80	S.F.D 50' x 100' 5,000 S.F.	485,758 S.F.	80	80	0	0
271, 272, 279, 281	4/124	CONDOMINIUM	671,128 S.F.	124	124	0	0
285–286	2/89	CONDOMINIUM	330,719 S.F.	89	89	0	0
273, 284	2/40	CONDOMINIUM	398,791 S.F.	120	0	120	0
276, 280, 287	3/42	CONDOMINIUM	788,159 S.F.	239	0	60	179
277, 278	2/27	AFFORDABLE CONDOMINIUM	444,636 S.F.	161	0	0	161
274	1/14	AFFORDABLE CONDO/APARTMENT (FOR LEASE)**	336,813 S.F.	140	0	0	140
275	1/15	CONDO/APARTMENT (FOR LEASE)**	479,250 S.F.	232	0	0	232
282, 283	2/14	CONDO/APARTMENT (FOR LEASE)**	160,132 S.F.	69	0	11	58
288,289	2/2	COMMERCIAL (FOR LEASE)**	82,206 S.F.	–	–	–	–
290	1/2	COMMERCIAL (FOR LEASE)**	100,942 S.F.	–	–	–	–
292–301	10/9	COMMERCIAL (FOR LEASE)**	763,848 S.F.	–	–	–	–
303–305	3/3	COMMERCIAL (FOR LEASE)**	147,490 S.F.	–	–	–	–
422	*1/0	PRIVATE DRIVEWAY & FIRE LANE	93,194 S.F.	–	–	–	–
306–308	3/3	RECREATION	253,595 S.F.	–	–	–	–
310	*1/NA	COMMUNITY PARK SITE	430,072 S.F.	–	–	–	–
309	*1/NA	SCHOOL SITE	423,951 S.F.	–	–	–	–
313–327, 344–346, 350, 352–353, 356–361, 363, 365–371, 374–421	*83/0	OPEN SPACE	1,040,689 S.F.	–	–	–	–
364	*1/0	PRIVATE PARK	24,053 S.F.	–	–	–	–
335, 336, 340	*3/0	SR 126 FUTURE R/W	407,000 S.F.	–	–	–	–
311, 329, 332, 333, 338, 342, 347, 351, 354, 355, 362, 372, 373	*13/0	OPEN SPACE (WATER QUALITY)	777,807 S.F.	–	–	–	–
339, 343	*2/0	OPEN SPACE (UTILITY CORRIDOR)	114,213 S.F.	–	–	–	–
331, 349	*2/0	OPEN SPACE (CL.1 BIKE TRAIL/ MAINT.ACCS.ROAD)	203,596 S.F.	–	–	–	–
330, 348	*2/0	OPEN SPACE (MULTI–USE TRAIL)	117,342 S.F.	–	–	–	–
334, 337, 341	*3/0	OPEN SPACE (LIGHT RAIL RESERVATION)	349,556 S.F.	–	–	–	–
302	*1/NA	FIRE STATION	58,404 S.F.	–	–	–	–
312	*1/NA	TRAIL HEAD	19,617 S.F.	–	–	–	–
291	1/NA	PARK & RIDE	35,143 S.F.	–	–	–	–
328	*1/NA	RECYCLED WATER BOOSTER PUMP STATION	1,226 S.F.	–	–	–	–
N/A	NA/NA	PUBLIC ROADS	2,072,022 S.F.	–	–	–	–
TOTAL	422		12,747,155 SF	1,444	483	191	770

* NO DEVELOPMENT PLANNED ON THESE LOTS
** TOTAL NUMBER OF LEASE RESIDENTIAL BUILDINGS = 43
TOTAL NUMBER OF LEASE COMMERCIAL BUILDINGS = 16

UNIT SUMMARY:

S.F.D. UNITS: – 270 UNITS
CONDOMINIUM UNITS: – 744
APARTMENTS UNITS: – 430
COMMERCIAL LOT ACREAGE: 25.13 ACCRES



FEET
600 300 0 600 1200
1 1/2 0 1 2
INCHES

PROJECT SUMMARY:

GROSS AREA – 292.6 Acres
TOTAL LOTS – 422
TOTAL D.U. – 1,444
EXISTING ZONING – SP (NEWHALL RANCH SPECIFIC PLAN)
PROPOSED ZONING – SP (NEWHALL RANCH SPECIFIC PLAN)
GENERAL PLAN LAND USE – SP (NEWHALL RANCH SPECIFIC PLAN)

GRADING SUMMARY

ONSITE
RAW CUT 343,000 CY
RAW FILL 5,590,000 CY
NET CUT 343,000 CY
NET FILL 6,263,000 CY
IMPORT 5,920,000 CY

OFFSITE (ADOBE CANYON)
RAW CUT 6,468,500 CY
RAW FILL 133,000 CY
NET CUT 6,468,500 CY
NET FILL 146,300 CY
EXPORT 6,322,200 CY

OFFSITE (CHIQUITO CANYON)
RAW CUT 1,190,000 CY
RAW FILL 1,071,000 CY
NET CUT 1,190,000 CY
NET FILL 1,190,000 CY
EXPORT 0 CY

OFFSITE (SOUTH BANK TEMP. LEVEE)
RAW CUT 0 CY
RAW FILL 250,000 CY
NET CUT 0 CY
NET FILL 316,000 CY
EXPORT 316,000 CY

OFFSITE (UTILITY CORRIDOR GRADING)
RAW CUT 3,200 CY
RAW FILL 233,000 CY
NET CUT 3,200 CY
NET FILL 263,000 CY
IMPORT 259,800 CY

OFFSITE (WRP DEBRIS BASINS)
RAW CUT 91,000 CY
RAW FILL 2,000 CY
NET CUT 91,000 CY
NET FILL 2,200 CY
EXPORT 88,800 CY

OFFSITE (ZONE 1A WATER TANK SITE)
RAW CUT 87,000 CY
RAW FILL 2,000 CY
NET CUT 87,000 CY
NET FILL 2,200 CY
EXPORT 84,800 CY

TOTAL EARTHWORK
RAW CUT 8,182,700 CY
RAW FILL 7,281,000 CY
NET CUT 8,182,700 CY
NET FILL 8,182,700 CY
IMPORT/ EXPORT: 0 CY

NOTE:
STATE ROUTE 126 (SR126) IS DESIGNATED BY LOS ANGELES COUNTY AS "HENRY MAYO DRIVE".

OFFSITE IMPROVEMENTS ASSOCIATED WITH VTTM 53108

1. LA COUNTY SANITATION DISTRICT TRUNK SEWER WESTERLY TO NEWHALL RANCH WRP
2. WATER MAIN EXTENSION TO VALENCIA WATER CO. STORAGE TANKS AND EXISTING PIPES
3. SEWER FORCE MAIN EASTERLY TO LA COUNTY SANITATION DISTRICT 32 WRP
4. BORROW SITE GRADING PER C.U.P. NO.200500112
5. DEBRIS BASINS NORTH OF SR 126
6. OFFSITE GRADING AS DEPICTED
7. OFFSITE IMPROVEMENTS FOR IRRIGATION WATER LINES AND WELLS

UTILITY PROVIDERS:

SEWER: L.A. COUNTY SANITATION DIST. 32 AND NEWHALL RANCH WRP
WATER: VALENCIA WATER COMPANY
ELECTRICAL: SOUTHERN CALIFORNIA EDISON
GAS: SOUTHERN CALIFORNIA GAS CO.
TELEPHONE: AT&T
CABLE TV: TIME WARNER CABLE

EASEMENTS:

EASEMENTS....... PLOTTABLE EASEMENTS ARE SHOWN FROM AN OWNER SUPPLIED TITLE REPORT. NON–PLOTTABLE ITEMS ARE NOTED BELOW AS INDETERMINATE OR DOES NOT AFFECT.

TITLE REPORT.... NORTH AMERICAN TITLE COMPANY REPORT NO. 7004810–23, DATED OCTOBER 31, 2007. THIS TITLE REPORT COVERS THIS PROPERTY.

(9) INDICATES TITLE REPORT EXCEPTION No

EXCEPTION NO.	OWNER	PURPOSE	REFERENCE	STATUS
2	COUNTY OF LOS ANGELES	PUBLIC ROAD & HIGHWAY	JUNE 27, 1910 INSTRUMENT NO. 47 BOOK 4230, PAGE 2, DEEDS	PLOTTED
3	COUNTY OF LOS ANGELES	PUBLIC ROAD & HIGHWAY	APRIL 6, 1920 INSTRUMENT NO. 621 BOOK 7171, PAGE 90, DEEDS	PLOTTED
6	STATE OF CALIFORNIA	PUBLIC HIGHWAY, SLOPES/DRAINAGE	JANUARY 12, 1939 DOCUMENT NO. 714H CERT. NO. JJ 84561, TORRENS	PLOTTED
7	AGREEMENT BETWEEN: NEWHALL LAND & FARMING CO. AUGUST A. & MARY J. RUBEL	SEE DOCUMENTS FOR FULL PARTICULARS	JUNE 14, 1949 INSTR. NO. 988 MODIFIED NOVEMBER 19, 1980 INSTR. NO. 80–1162907	INDETERMINATE
9	BARNSDALL OIL COMPANY	PIPE LINES	DEC. 4, 1944 INSTR. NO. 1236 AMENDED DECEMBER 4, 1944 INSTR. NO. 1236, O.R.	PLOTTED
13	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES	APRIL 10, 1951 INSTRUMENT NO. 2549 BOOK 36011, PAGE 33, O.R.	INDETERMINATE
14	UNION OIL COMPANY OF CALIFORNIA	PIPE LINES & APPURTENANCES	JUNE 19, 1951 INSTRUMENT NO. 2670 BOOK 36565, PAGE 89, O.R.	INDETERMINATE
15	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES	MAY 9, 1952 INSTRUMENT NO. 2456 BOOK 38895, PAGE 359, O.R.	INDETERMINATE
16	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES	DECEMBER 21, 1953 INSTRUMENT NO. 1702 BOOK 43431, PAGE 69, O.R.	INDETERMINATE
17	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES	DECEMBER 21, 1953 INSTRUMENT NO. 2788 BOOK 43504, PAGE 347, O.R.	INDETERMINATE
18	UNION OIL COMPANY OF CALIFORNIA	POWER POLES & APPURTENANCES	JUNE 13, 1958 INSTRUMENT NO. 3434 BOOK D127, PAGE 71, O.R.	PLOTTED
20	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES	NOVEMBER 25, 1958 INSTRUMENT NO. 3316 BOOK D286, PAGE 279, O.R.	INDETERMINATE
21	UNION OIL COMPANY OF CALIFORNIA	PIPE LINES & APPURTENANCES	AUGUST 29, 1961 INSTRUMENT NO. 3943, O.R.	INDETERMINATE
23	UNION OIL COMPANY OF CALIFORNIA	PIPE LINES	FEBRUARY 7, 1962 INSTRUMENT NO. 3210 BOOK D1505, PAGE 463, O.R.	PLOTTED
24	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRIC LINES, INGRESS, EGRESS	JULY 23, 1964 INSTRUMENT NO. 5047, O.R.	DOES NOT AFFECT PROPERTY
26	HUMBLE OIL AND REFINING COMPANY	PIPE LINES, ACCESS ROADS, WELLS	MARCH 9, 1970 INSTRUMENT NO. 1741 BOOK M3439, PAGE 696, O.R.	INDETERMINATE
27	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRICAL & COMMUNICATION SYSTEMS	MAY 27, 1971 INSTRUMENT NO. 3947 BOOK D5071, PAGE 131, O.R.	INDETERMINATE
28	TEXACO INC.	PUMP STATION, PIPE LINES, & APPURTENANCES	OCTOBER 14, 1971 INSTRUMENT NO. 3990 BOOK D5223, PAGE 940, O.R.	PLOTTED
30	SOUTHERN CALIFORNIA EDISON COMPANY	ELECTRICAL & COMMUNICATION SYSTEMS	DECEMBER 2, 1971 INSTRUMENT NO. 2421 BOOK D5275, PAGE 682, O.R.	INDETERMINATE
31	EXXON CORPORATION	PIPE LINES & APPURTENANCES	JUNE 21, 1979 INSTRUMENT NO. 79–675839, O.R.	PLOTTED
33	THE PACIFIC TELEPHONE AND TELEGRAPH COMPANY	POLE LINES & APPURTENANCES	MARCH 6, 1981 INSTRUMENT NO. 81–2236554, O.R.	PLOTTED
34	TEXACO CALIFORNIA PIPELINES, INC.	PIPE LINE & PUMP STATION	SEPTEMBER 8, 1993 INSTRUMENT NO. 93–1746382, O.R.	INDETERMINATE
36	STATE OF CALIFORNIA	TRAFFIC CONTROL DEVICES	MARCH 21, 1997 INSTRUMENT NO. 97–434774, O.R.	PLOTTED
37	THE NEWHALL LAND AND FARMING COMPANY	WATER PIPELINES PUMP STATIONS, & APPURTENANCES	DECEMBER 23, 1999 INSTRUMENT NO.s 99–2360380 & 99–2360381, O.R. DECEMBER 28, 1999 INSTRUMENT NO. 99–2375698, O.R. MARCH 17, 2000 INSTRUMENT NO.s 00–399620, 00–399621, 00–399622, 00–399623, 00–399624, 00–399625, O.R.	PLOTTED
38	County of Los Angeles	INGRESS & EGRESS RESTRICTIONS	PARCEL MAP NO. 24500–01 P.M.B. 293–34–67	PLOTTED
39	LONG CANYON ROAD	FUTURE STREET	PARCEL MAP NO. 24500–01 P.M.B. 293–34–67	PLOTTED
47	STATE OF CALIFORNIA	SLOPES & DRAINAGE	SEPTEMBER 15, 2000 INSTRUMENT NO. 00–1445425, O.R.	PLOTTED

LEGEND:

100 – LOT NO.
5000 SF – LOT AREA (SQUARE FEET)
1210.5 – PAD ELEVATION
O.S. OPEN SPACE
P.F. PUBLIC FACILITY
PROJECT BOUNDARY
EXISTING RIGHT OF WAY
PROPOSED EASEMENT
EXISTING EASEMENT
PROPOSED RIGHT OF WAY
950 PROPOSED CONTOUR
DAYLIGHT LINE
SD STORM DRAIN
SS SANITARY SEWER (SERVICE LOCAL)
SST SANITARY SEWER (TRUNK SEWER)
FM SANITARY SEWER (FORCE MAIN)
W WATER
RW RECLAIMED WATER
IRRI IRRIGATION LINE
W EXISTING WATER LINE
E EXISTING POWER LINE
COM EXISTING COMMUNICATION LINE
VIREO_HABITAT EXISTING VIREO HABITAT LINE
EXISTING CMP INLET
CATCH BASIN
WATER GATE VALVE
RC03 RIVER CORRIDOR–2003
FEMA FEMA
X–Q50 EXISTING Q–CAP
RS RESOURCE LINE
ACOE ARMY CORPS OF ENGINEERS JURISDICTION LINE
OIL WELL GPS POSITIONED
OIL WELL DEPT. OF OIL AND GAS CALCULATED POSITION
OIL WELL DIGITIZED POSITION
OIL WELL DEPT. OF OIL AND GAS METAL DETECTOR POSITION
COAST LIVE OAK (Quercus agrifolia)
VALLEY OAK (Quercus lobata)
EXISTING TREE CANOPY
EXISTING POWER POLE
W.S. WATER SURFACE
BIKE TRAIL/ LACFCD ACCESS ROAD/LACSD ACCESS ROAD
LA COUNTY RIDING/HIKING TRAIL
BURIED BANK STABILIZATION
SIGHT DISTANCE LINE
RETAINING WALL
SOUND WALL

SURVEY NOTES:

VERTICAL DATUM:
LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS BENCHMARK DESCRIBED AS FOLLOWS: LACDPW L5402 RND HD SPK IN LOWER CONC HDWL @ N END 24 FT W/O C/L THE OLD RD (W RDWY) AND 0.6 MI S/O HENRY MAYO DR @ MI MKR #6.25 NEWHALL QUAD, 1995 ADJUSTMENT (NAVD 1988) ELEVATION = 1031.951

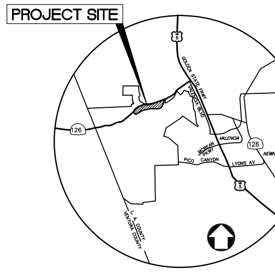
THERE IS APPROXIMATELY 2.3 FEET DIFFERENCE BETWEEN PREVIOUS COUNTY OF LOS ANGELES DATUMS BASED UPON NAVD 1929 AND NAVD 1988 (NAVD 1988 IS 2.3 FEET HIGHER THAN NAVD 1929).

AERIAL PLANIMETRY:
THIS SURVEY WAS PREPARED BY PHOTOGRAMMETRIC PROCESSES, UTILIZING AERIAL PHOTOGRAPHY DATED NOVEMBER 15, 2006. THE OUTLINE OF BUILDINGS AND OTHER ROOFED IMPROVEMENTS, MAY INCLUDE EAVES, CANOPIES AND BALCONIES, SINCE BUILDINGS BELOW THESE ITEMS ARE NOT VISIBLE TO THE CAMERA.

TOPOGRAPHY:
BY PHOTOGRAMMETRIC PROCESSES FROM LIDAR IMAGING DATED NOVEMBER 15, 2006, AND COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS.

CONTOUR INTERVAL:
10 FOOT, UNLESS OTHERWISE NOTED.

PROJECT SITE



VICINITY MAP

NOT TO SCALE

GENERAL NOTES:

1. GRADE ELEVATIONS ON THE VTTM ARE APPROXIMATE. CHANGES IN THE ELEVATIONS DEPICTED ON THE TENTATIVE MAP WHICH WILL NOT RESULT IN PAD ELEVATION CHANGES OF MORE THAN 5 FEET ARE PERMITTED SUBJECT TO THE PROVISIONS OF SECTION 5.2 OF THE NEWHALL RANCH SPECIFIC PLAN, WHICH MAY REQUIRE AN AMENDED MAP.
2. PERMISSION IS REQUESTED FOR LOT LINES TO BE ADJUSTED TO THE SATISFACTION OF DRP AND DPW PROVIDED NO ADDITIONAL DEVELOPABLE LOTS ARE CREATED. THE DEGREE OF ADJUSTMENT SHALL BE CONSISTENT WITH THE INTENT OF THE SUBDIVISION MAP APPROVAL AND THE SUBDIVISION MAP ACT.
3. ALIGNMENT AND GEOMETRICS OF STREETS AND TRAILS ARE APPROXIMATE. PERMISSION IS REQUESTED FOR ADJUSTMENTS TO STREETS TO BE MADE PROVIDED THE DEPARTMENT OF PUBLIC WORKS AND DRP DETERMINE THAT THE ADJUSTMENTS ARE CONSISTENT WITH THE INTENT OF THE SUBDIVISION APPROVAL. ADJUSTMENTS TO TRAILS CAN BE MADE WITH APPROVAL BY DRP, DPW AND THE DEPARTMENT OF PARKS AND RECREATION.
4. BUILDING FOOTPRINTS SHOWN ON THE TENTATIVE MAP OR ACCOMPANYING SITE PLAN ARE ONLY FOR ILLUSTRATIVE PURPOSES. RELOCATION OF BUILDINGS, OR ADJUSTMENTS IN BUILDING SQUARE FOOTAGE, NUMBER OF BUILDINGS, PARKING REQUIREMENTS AND OTHER FEATURES WHICH WILL NOT RESULT IN AN INCREASE IN TOTAL SQUARE FOOTAGE OR THE NUMBER OF DWELLING UNITS ARE PERMITTED SUBJECT TO THE PROVISIONS OF SECTION 5.2 OF THE NEWHALL RANCH SPECIFIC PLAN.
5. PERMISSION IS REQUESTED TO FILE "LARGE LOT" PARCEL MAPS OF 20 ACRES OR MORE (WITHOUT IMPROVEMENTS) OR 5 ACRES OR MORE (WITH IMPROVEMENTS) AS THE FIRST UNIT FINAL MAP. RECORDATION OF A LARGE LOT PARCEL MAP SHALL BE CONSIDERED THE FILING OF THE FIRST UNIT MAP FOR THE PURPOSES OF TIME EXTENSIONS PUSUANT TO THE SUBDIVISION MAP ACT BUT SHALL NOT REQUIRE THE OBLIGATION OF FEES, DEDICATIONS (EG. RIGHTS OF WAY OR PARKS) EXACTIONS OR INFRASTRUCTURE EXCEPT AS MAY BE REQUIRED BY CHAPTER 21.32 OF THE SUBDIVISION MAP ACT.
6. PERMISSION IS REQUESTED FOR UNIT MAP PHASING.
7. PERMISSION IS REQUESTED TO COMBINE LOTS.
8. PERMISSION IS REQUESTED TO CREATE AND RECORD ADDITIONAL OPEN SPACE LOTS.
9. PERMISSION IS REQUESTED TO RECORD ADDITIONAL UTILITY LOTS AND PROVIDE MAINTENANCE EASEMENTS TO THE SATISFACTION OF THE DRP AND DPW.
10. THE LOCATIONS OF APPURTENANT STRUCTURES (E.G., PASEOS, PEDESTRIAN BRIDGES, TRANSIT SHELTERS) MAY BE RELOCATED IF DETERMINED TO BE CONSISTENT WITH THE APPROVED TENTATIVE MAP BY DRP AND DPW.
11. REQUEST PERMISSION TO PHASE MASS GRADE TO THE SATISFACTION OF DRP AND DPW AND THE PROVISIONS OF THE COUNTY CODE, APPROVED PROJECT CONDITIONS AND MITIGATION MEASURES.
12. PROPOSED STREET GRADES ARE APPROXIMATE AND SUBJECT TO ADJUSTMENTS PENDING DETERMINATION OF FINAL DEVELOPMENT LAYOUT AND PLANS. ANY CHANGES SHALL BE TO THE SATISFACTION OF DPW.
13. LOCATION OF TRAILS TO BE FINALIZED PRIOR TO FINAL MAP APPROVAL.
14. PROVIDE PROPERTY LINE RETURN RADII OF 13 FEET AT ALL LOCAL STREET AND PRIVATE DRIVE INTERSECTIONS, AND 27 FT AT THE INTERSECTION OF LOCAL STREETS OR PRIVATE DRIVES WITH PLANNED HIGHWAYS (THOSE ON THE COUNTY HIGHWAY PLAN) AND WHERE ALL PLANNED HIGHWAYS INTERSECT OR WHERE ONE OF THE ROADS SERVES A COMMERCIAL OR INDUSTRIAL DEVELOPMENT PLUS ADDITIONAL RIGHT OF WAY FOR CORNER CUT OFF TO MEET CURRENT GUIDELINES OF THE AMERICANS WITH DISABILITIES ACT (ADA) TO THE SATISFACTION OF PUBLIC WORKS.
15. MODIFIED STREET CROSS–SECTIONS AS SHOWN ON VESTING TENTATIVE TRACT MAP ARE REQUESTED. HOWEVER, PERMISSION IS SOUGHT TO CONSTRUCT STANDARD AND ALTERNATE STREET CROSS–SECTIONS AT THE DISCRETION OF THE SUBDIVIDER.
16. ALL DIMENSIONS SHOWN ARE APPROXIMATE.
17. TOPO COMPILED 11/15/06 BY HUNSAKER FROM DATA FLOWN 5/24/06.
18. ADDITIONAL RIGHT OF WAY WILL BE DEDICATED AS REQUIRED AT ALL PROPOSED ROUNDABOUT LOCATIONS ALONG "A" STREET AND ALONG WOLCOTT ROAD TO THE SATISFACTION OF PUBLIC WORKS.
19. AS A DEDICATION TO PUBLIC USE, WHILE ALL OF LONG CANYON ROAD, WOLCOTT ROAD, AND "A" STREET WITHIN THIS SUBDIVISION REMAINS A PUBLIC STREET, WE HEREBY GRANT TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT DIRECT VEHICULAR INGRESS AND EGRESS TO SAID STREET.
20. RELATED ENTITLEMENTS:
    - SEA/SMA, PROJECT & OFF–SITE GRADING CUP 00–196
    - GPA 00–196 (MASTER PLAN OF HIGHWAYS)
    - LPA 00–196 (CIRCULATION PLAN)
    - SPA 00–196 (MASTER CIRCULATION PLAN & TEXT)
    - OAK TREE PERMIT 00–196
21. PERMISSION IS REQUESTED TO ALLOW OFFICE & RETAIL SPACE TO BE INTERCHANGEABLE PROVIDED THE REQUIRED PARKING IS SATISFIED.
22. PERMISSION IS REQUESTED TO ADJUST LOT LINES BETWEEN RESIDENTIAL OR COMMERCIAL LOTS AND OPEN SPACE LOTS SUBSEQUENT TO ANY UNIT MAP RECORDATION PROVIDED THE TOTAL OPEN SPACE IS EQUAL TO OR GREATER THAN 30%.
23. REQUEST MODIFICATION OF STREET FRONTAGE REQUIREMENTS PER SECTION 21–24–040 OF THE L.A. COUNTY CODE.
24. SHARED AND RECIPROCAL PARKING IS DEDICATED PER THE PROVISIONS OF THE NEWHALL RANCH SPECIFIC PLANS SUBJECT TO CONFORMANCE REVIEW OR APPROVAL OF A PARKING PROGRAM.
25. ALL DETAILS RELATED TO THE CONSTRUCTION OF ALTERNATIVE WATER QUALITY DEVICES, SUCH AS WATER QUALITY SWALES, ARE SUBJECT TO REVIEW DURING THE IMPROVEMENT PLAN STAGE.
26. ALL STREETS ARE PUBLIC STREETS UNLESS NOTED "PRIVATE DRIVEWAY AND FIRE LANE".
27. LOCATION AND DETAILS OF WATER QUALITY DEVICES AND DEBRIS BASINS ARE APPROXIMATE. PERMISSION IS REQUESTED TO ADJUST DESIGNS OF THESE FACILITIES PROVIDED DPW AND DRP DETERMINE THAT THE ADJUSTMENTS ARE CONSISTENT WITH THE INTENT OF THE SUBDIVISION APPROVAL.

NOTE:
THE ABOVE MENTIONED GENERAL NOTES (THAT ADDRESS MINOR MODIFICATION) MAY REQUIRE REVIEW AND APPROVAL OF A REVISED EXHIBIT MAP.

SPECIAL NOTES:

1. WATER QUALITY BASINS AND BIOSWALES WILL PROVIDE FOR TREATMENT OF STORM WATER RUN OFF PER THE APPROVED DRAINAGE CONCEPT.
2. THE WATER QUALITY BASINS SHALL BE DEDICATED TO THE COUNTY WHEN COMPLETED AND ACCEPTED.
3. PROPOSED RECLAIMED WATER LINES WILL BE LOCATED IN PUBLIC STREETS AND TRAILS.
4. THE FOLLOWING LOTS ARE SUBJECT TO R/W DEDICATION FOR FUTURE SR–126 IMPROVEMENTS: 335, 336 AND 340.
5. ACCESS WILL BE PROVIDED TO OPEN SPACE LOTS 328,330–333,348–349,351, AND 354–356 BY ACCESS EASEMENTS THROUGH ADJOINING LOTS.
6. ACCESS WILL BE PROVIDED TO COMMERCIAL LOTS 292–301, AND 303–305 BY RECIPROCAL ACCESS EASEMENTS THROUGH ADJOINING LOTS.
7. PROPOSED PRIVATE AGRICULTURAL WATER FACILITIES WILL BE LOCATED WITHIN OPEN SPACE LOTS ACCESSABLE BY TRAILS AND ACCESS DRIVES. EASEMENTS WILL BE RESERVED FOR THESE FACILITIES IN ANY LAND TRANSACTIONS.

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:
PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500–01 PMB 293–34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28480 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 219–6000 (661) 775–2718 (FAX)
ERIK WINATA R.C.E. NO. C 67285 DATE

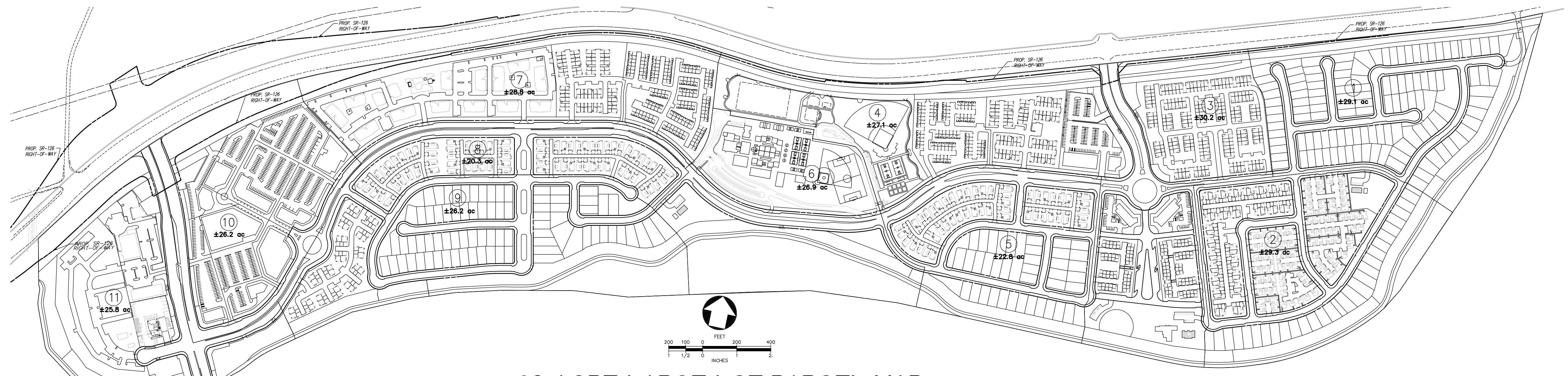
DESIGNED : NSK/YL
DRAFTED : TLC/JS
CHECKED : EW

OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255–4000
REPRESENTATIVE : MR. FRED MACMURDO

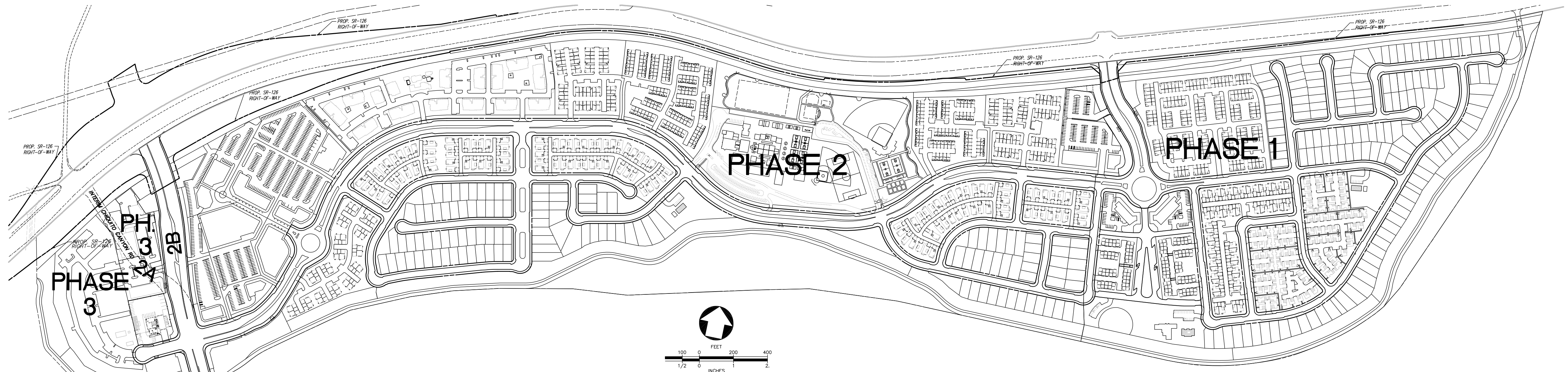
MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 1 OF 13

Plotted – 12/17/2009 2:44:08 PM :: Saved – 12/15/2009 8:45:27 AM



20 ACRE LARGE LOT PARCEL MAP

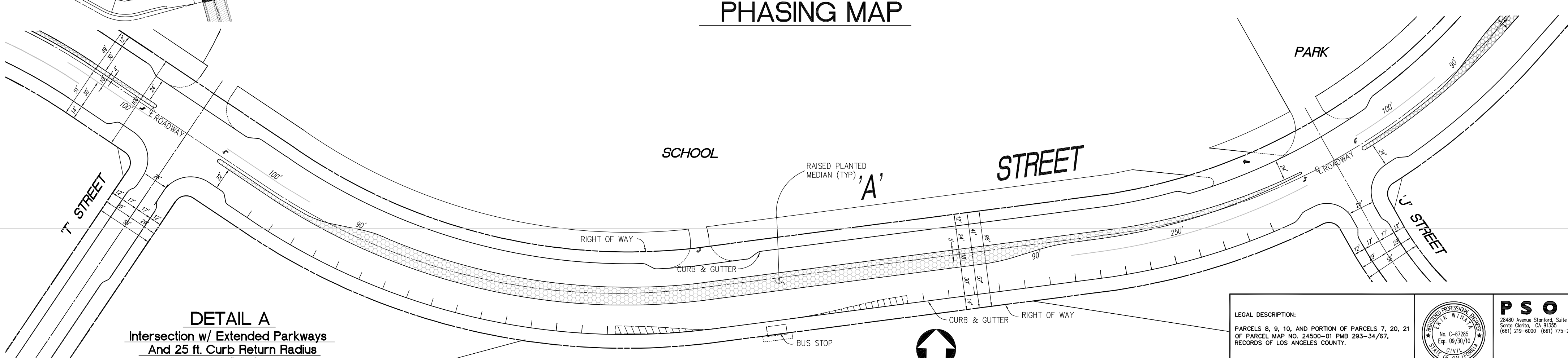


PHASING MAP

PHASE 1	
Total Ac	89.64 AC
Total Du	502 DU
SFD	149 DU
Condo's	353 DU
Total Commercial Sf	30,800 SF
Total Open Space	27.07 AC
Park and Ride	0.81 AC

PHASE 2	
Total Ac	184.95 AC
Total Du	802 DU
SFD	121 DU
Condo's	681 DU
Total Commercial Sf	592,400 SF
Total Open Space	40.83 AC
Public Park	11.26 AC
School	9.73 AC

PHASE 3	
Total Ac	18.08 AC
Total Du	140 DU
SFD	0 DU
Condo's	140 DU
Total Commercial Sf	29,250 SF
Total Open Space	6.66 AC
Fire Station	1.34 AC



DETAIL A

Intersection w/ Extended Parkways And 25 ft. Curb Return Radius

SCALE 1" = 40'

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

LEGAL DESCRIPTION:

PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28480 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED : NSK/YL
DRAFTED : TLC/JS
CHECKED : EW

OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

MAJOR LAND DIVISION VESTING TENTATIVE TRACT MAP NO. 53108

LARGE LOT CONVEYANCE MAP, PHASING MAP & DETAIL PLAN
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 13 OF 13

RV001-13.dwg

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

February 25, 2010 Job No. 10-1702R-4

The Newhall Land and Farming Company
25124 Springfield Court, Suite 300
Valencia, CA 91355

Attention: Mr. Fred MacMurdo

Subject: **GEOLOGIC AND GEOTECHNICAL REPORT – ADDENDUM NO. 1**
Response to Los Angeles County Review Sheets (dated 2/5/10)

Project: Vesting Tentative Tract 53108, Map dated 12/21/09
Landmark Village
Newhall Ranch
Los Angeles County, California

References: See end of text

Dear Mr. MacMurdo:

This addendum report has been prepared in response to Geologic and Soils Engineering Review Sheets (dated February 5, 2010), prepared by the Los Angeles County Department of Public Works Geotechnical and Materials Engineering Division regarding our December 21, 2009 report for the Revised Map for Vesting Tentative Tract 53108, also dated December 21, 2009. In order to facilitate the review process, we are presenting the County remarks (**in bold**) in the order in which they appear on the original review letter. Each remark is followed immediately by our response. A response to Remark No. 6 of the Soils Engineering Review Sheet is provided by R.T. Frankian and Associates in a Letter Report (attached as **Appendix B**). A copy of these review sheets is attached after the references.

GEOLOGIC REVIEW SHEET DATED 2/5/10

Remark No. 1

The base used for the geologic map does not conform to the most recent tentative map submitted through the Department of Regional Planning. This review is based solely on the geologic map, and additional review comments may be provided after receipt of a revised tentative map.

Response

Acknowledged. A revised VTTM will be formally submitted through the Subdivision Committee process within the next few months.

Remark No. 2

Show proposed final topography for landslides to be completely or partially removed.

Response

No landslide removals are proposed to extend beyond the current limits of grading shown on the Tentative Tract Map at the offsite Chiquito Canyon Disposal Site areas. Potentially unstable landslides that will not be removed will be placed in Restricted Use Areas. We have provided additional details in response to this remark in our response to Comment No. 2 of the Soils Engineering Review Sheet.

Remark No. 3

The proposed cut slope on sheet 11 above Trench T-23L may expose a daylighted component of bedding. Provide a geologic cross section(s) and stability analyses for this slope, and any mitigation recommendations as necessary. Note: Grading Plan review will require continuous stratigraphy be provided to further address stability of this cut.

Response

The Tentative Tract Map submitted with our 12/21/09 report will be revised prior to formal submittal through the Subdivision Committee process to eliminate fill portions of the subject slope proposed at Adobe Canyon (attached Exhibit **Figure 1, Appendix A** reflects the intended modification to be incorporated in the formal VTTM submittal at a later date). Bedding attitudes measured at this slope indicate that the average true dip is 40° to the northeast, which is substantially steeper than the proposed 2.5:1 cut slope gradient. Cross Section 1A-1A' illustrates the anticipated conditions on the east-facing portion of the proposed cut slope. Bedding with an apparent dip of 24° will be exposed in this portion of the slope, which is steeper than the proposed 22° slope gradient. In addition, this portion of the slope is more than 45° off of the bedding dip direction, which is considered an extreme bedding component; therefore the bedding is interpreted as being generally neutral to the

slope face. This geometry is considered globally stable from a geologic standpoint and based on stability analysis of higher 2:1 cut slopes proposed at Adobe Canyon (see analysis of Cross Section 28L-28L' in our 12/29/09 report). The more southeasterly facing portion of the slope will expose neutral bedding conditions. In conclusion, the subject cut slope will expose bedding that is generally neutral to the slope or steeper than the proposed slope and is therefore considered globally stable from a geologic standpoint. This slope should be geologically observed during construction to assess if there is a potential for shallow wedge-type failures along weak bedding planes, which can be mitigated as needed with stability fills.

Remark No. 4

The Soils Engineering review dated 2/5/10 is attached.

Response

Our response to the Soils Engineering review sheet is presented below.

SOILS ENGINEERING REVIEW SHEET DATED 2/5/10 (09)

Remark No. 1

The geotechnical map within the submitted report does not match the latest approved Tentative Tract Map dated by Regional Planning 5/2/07. Provide Revised Tentative Tract Map dated by Regional Planning and geotechnical map that conform.

Response

Acknowledged. A revised VTTM will be formally submitted through the Subdivision Committee process within the next few months.

Remark No. 2

Show the final graded contours of all landslides to be removed and recompacted on the geotechnical maps and Tentative Map. For example, Landslide Qls XVI (B) is recommended to be removed and recompacted, however, the final graded contours are not shown on the geotechnical map or the Tentative Map. Revise geotechnical maps and Tentative Map as necessary.

Response

Per our discussions with Newhall Land, landslides mapped within and adjacent to the proposed fill disposal site areas north of Highway 126 will not be mitigated as part of the Landmark Village Development except at Landslide Qls-XVIB as described below. Therefore, grading is not anticipated to extend beyond the limits of grading on this Tentative Tract map. The disposal fills are generally proposed over the toe portions of the landslides, which will improve stability by adding resisting force to potential movement. The footprint of disposal site DF-4 has been modified to avoid impacting Landslide Qls-XIB (the attached Exhibit, **Figure 2, Appendix A** reflects the intended modification to be incorporated in the formal VTTM submittal at a later date). Restricted Use Areas have been recommended around these landslides on the attached geologic/geotechnical maps. Mitigation of these landslides was addressed in our September 30, 2005 Geologic/Geotechnical Report for future development of the Homestead Project, Vesting Tentative Tract 060678.

The limits of Landslide Qls-XVIB have been revised into two surficial failures based on additional review of Trench Logs T-110 and T-125, regional bedding structure, and geomorphic expression, as shown on the attached geologic/geotechnical maps (see attached Geologic/Geotechnical Map Sheet 11 of 13). Cross Section 2A-2A' has been prepared illustrating the anticipated geologic conditions oriented in the down-dip direction of bedding and in the likely direction of past movement of the lower surficial failure. Stability analysis of both the surficial failure and the underlying bedding conditions adjacent to the proposed debris basin indicate factors of safety in excess of minimum County requirements (see **Appendix A** for analysis). The portion of the landslide below the proposed debris basin should be removed and replaced with compacted fill to provide competent subgrade materials for the basin. The upper surficial failure will mostly be removed by cut-slope CS-3. The remaining portion of this failure is over 150 ft. from the proposed debris basin and, therefore, will not have any adverse impact. The remaining surficial failure material will be mitigated during future development of the Homestead project, Vesting Tentative Tract 060678.

Remark No. 3

On the geotechnical map, delineate approximately limits of proposed disposal fills. Also delineate all soils unsuitable for the support of structures (proposed disposal fill, landslide debris) and prominently note/label with the note "Unsuitable Soils-Not Suitable for the Support of Structures".

Response

The limits of the proposed disposal fills have been delineated on the attached geotechnical map and on the revised design for disposal site DF-4 (**Figure 2**). Based on discussions with Newhall Land, unsuitable landslide material and alluvial soils will not be mitigated for the intended use as disposal fills for Vesting Tentative Tract 53108 except as described in our responses to remarks No. 2 and No. 4. Unsuitable landslide, slopewash, and alluvial materials should be removed below the proposed fill slopes and adjacent debris basins at disposal sites DF-1 and DF-2 to provide stable, permanent slope conditions and support for the basins. The limits of this structural fill and of the soils not suitable for the support of structures have been prominently noted on the attached geotechnical map.

Remark No. 4

Discuss and address whether the proposed fill slopes associated with Disposal Fill areas will be subject to erosion and/or slope instability. Provide updated surficial, static, and seismic slope stability analyses considering the density, moisture content, and shear strengths of the disposal fill materials as well as the proposed design shown on the latest geotechnical map and Tentative Map. Provide additional recommendations as necessary.

Response

The fill placed in slope areas of the proposed disposal sites should be placed at a minimum 90% relative compaction. Specifically, fill within 20 feet horizontal of the proposed 4:1 slope face should be placed as a compacted and tested fill at Disposal Sites DF-3 and DF-4. The fill below and for 100 ft behind the 2:1 fill slope at Disposal Site DF-4 should be compacted to provide surficially and globally stable conditions. The fill placed for the proposed fill slopes and debris basins adjacent to Highway 126 at Disposal Sites of DF-1 and DF-2 should be placed as compacted fill and should be keyed into competent material to

provide both global and surficial stability. The anticipated removal depths are noted on the attached Sheet 11 of 13. Cross Section 3A-3A' illustrates the critical fill slope condition with uncompacted fill behind the slope. Stability analysis of this slope indicates factors of safety in excess of minimum County requirements for both static and pseudostatic conditions (see **Appendix A** for analysis). Based on previous analyses for VTT 060678 (see referenced report) compacted fill slope faces in this area exceed minimum County requirements for surficial stability. Owing to the gentle slope gradient, 4:1 slopes are anticipated to be globally stable with a compacted fill blanket overlying uncompacted fill.

Remark No. 5

Proposed Disposal Fill areas DF-1 and DF-2 appear to show debris basins located in disposal fill. Clarify the soil material that will be used to construct the proposed debris basins and provide recommendations as necessary.

Response

The disposal fill materials proposed below the debris basins are recommended to be placed at a minimum 90% relative compaction. It is also recommended that the landslide debris slopewash, alluvium and existing artificial fill be removed below the footprint of the proposed debris basins. Both debris basins will therefore be founded in compacted fill.

Remark No. 6

Provide static and seismic slope stability analyses for the proposed cut slopes surrounding the Zone 1A Water Tank at PM 18108. The referenced R.T. Frankian Report dated 3/28/08 does not include slope stability analyses. Also, provide a geotechnical cross section, for each section analyzed, showing the critical failure plane used in the analyses. Indicate the various shear strength parameters used in the analyses, in the appropriate segments of each failure plane. Show locations of the cross sections used in the slope stability analyses on the geotechnical map. Recommend mitigation if factors of safety are below County minimum standards.

Response

A response letter addressing this comment has been provided by the project geotechnical engineer for the tank site in the report by R.T. Frankian & Associates and is attached in **Appendix B** for review. We have reviewed this report and accept that the findings, conclusions and recommendations presented therein are acceptable for the proposed use.

Remark No. 7

The submitted report states that the design of the bank protection has been modified. Provide clarification, additional discussion, and details in regards to the stated design modification in relation to proposed bank protection shown on both the North and South banks of the Santa Clara River. The proposed Bank Stabilization is necessary for the feasibility of the proposed tract development and therefore must be addressed at the Tentative Map stage. Clarify design and provide additional information as necessary.

Response

The design presented on the Tentative Tract Map for Vesting Tentative Tract 53108 addressed in our previous report dated 9/27/00 consisted of a 2:1 (h:v) slope composed of either riprap, gunite or soil cement with an outboard 4:1 fill slope. The design on the Revised Tentative Map shows soil cement constructed at 1.5:1 to 2:1 gradient with a 3:1 outboard fill slope, both at Vesting Tentative Tract 53108 and at the onion fields site on the south side of the river. Cross Section 4A-4A illustrates the typical geometry for these slopes. Slope stability analysis of this Cross Section indicates that the proposed soil cement slope will exceed minimum County Factors of Safety for global static and pseudostatic slope stability. The slope also passes rapid draw down stability analysis (see **Appendix A** for stability analysis). A separate report addressing details of the proposed bank protection design and construction should be prepared when detailed plans are available for review.

Remark No. 8

Show the following on the geotechnical map and Tentative Map:

a. Final graded contours for the removal and recompaction of existing landslides.
b. Delineate the location of the proposed disposal fill materials.

c. Delineate the approximate limits of all soils unsuitable for the support of structure and prominently note/label with the note: "Unsuitable Soils – Not suitable for the Support of Structures".

d. All recommended mitigation measures.

Response

a. No landslide removals are proposed to extend beyond the grading footprint shown on the Tentative Map and in **Figure 2**. The Tentative Map will be modified by Psomas to incorporate the revised design for Disposal Site DF-4.
b. The locations of the proposed disposal fill materials are delineated on the attached Sheet 11 of 13 and on Figure 2. the Tentative Map will be modified by Psomas to incorporate the revised design for Disposal Site DF-4 into the Tentative Map and to delineate the locations of each disposal fill.
c. The limits of all soils unsuitable for the support of structures have been prominently labeled on Sheet 11 of 13 and on Figure 2. The Tentative map will be modified by Psomas to incorporate these limits.
d. Recommended mitigation measures for Landslide removals, alluvial removals, Restricted Use areas, and slope construction are noted on Sheet 11 of 13 and on Figure 2. The Tentative Map will be modified by Psomas to incorporate these notes into the Tentative Map.

Remark No. 9

Requirements of the Geology Section are attached.

Response

Responses to the Geological Review Sheet are provided in the first section of this report.

Remark No. 10

Include a copy of this review sheet with your response.

Response

A copy of this review sheet is attached after the references.

This opportunity to be of service is greatly appreciated. If you have any questions regarding this report, please call.

Respectfully submitted,

Brian J. Swanson, CEG 2055
Associate Geologist

Kevin P. Callahan, PE 72202
Professional Engineer

Reviewed by:

Eric J. Seward, CEG 2110
Principal Engineering Geologist
Vice President

The following attachments and appendices complete this report.

References
Geologic Review Sheet dated 2/5/10
Soils Engineering Review Sheet dated 2/5/10 (09)

APPENDIX A

- Geologic Cross Sections 1A-1A' through 4A-4A'
- Slope Stability Analyses
- Geologic/Geotechnical Map Sheet 11 of 13
- Exhibit Map – Revised Cut Slope at Adobe Canyon Figure 1
- Exhibit Map – Revised Design for Disposal Site DF-4 Figure 2

APPENDIX B

- Response to Soils Comment No. 6 Report by R.T. Frankian & Associates (dated 2/23/10)

Distribution: (1) The Newhall Land and Farming Company
Attn: Mr. Fred MacMurdo
(1) Psomas
Attn: Mr. Ross Barker
(2) County of Los Angeles Department of Public Works Geotechnical and Materials Engineering Division (Hardcopy and Electronic Copy)
Attn: Soils Section
Attn: Geology Section

REFERENCES

1. **GEOLOGIC/GEOTECHNICAL REPORT**
Vesting Tentative Tract 53108, dated 6/11/00
River Village, Newhall Ranch
Castaic, California
Dated September 27, 2000 – JN: 00-1702R-4

2. **GEOLOGIC AND GEOTECHNICAL REPORT – ADDENDUM NO. 1**
Response to County Comments (Review Sheets dated 12/12/00 and 1/2/01)
Vesting Tentative Tract 53108, Map Dated 6/11/00
River Village, Newhall Ranch
Castaic, California
Dated February 10, 2001 – JN: 01-1702R-4

3. **GEOLOGIC AND GEOTECHNICAL REPORT**
Review of Vesting Tentative Tract Map 060678 (Dated 8/3/05)
Vesting Tentative Tract 060678
Homestead
Newhall Ranch, California
Dated September 30, 2005 - JN: 05-2062-4

4. **GEOLOGIC AND GEOTECHNICAL REPORT**
Review of Revised Vesting Tentative Tract Map 53108 (Dated 12/21/09)
Landmark Village
Newhall Ranch
Los Angeles County, California
Dated December 21, 20096 - JN: 09-1702R-4

Sheet 1 of 1

County of Los Angeles Department of Public Works
GEOTECHNICAL AND MATERIALS ENGINEERING DIVISION
GEOLOGIC REVIEW SHEET
900 So. Fremont Ave., Alhambra, CA 91803
TEL. (626) 458-4925

DISTRIBUTION
1 Geologist
__ Soils Engineer
1 GMED File
1 Subdivision

TENTATIVE TRACT MAP 53108
SUBDIVIDER Newhall Land & Farming Co.
ENGINEER Psomas
GEOLOGIST & SOILS ENGINEER Allan Seward

TENTATIVE MAP DATED 5/2/07 (Rev.)
LOCATION Newhall Ranch

REPORT DATE 12/21/09, 10/29/03, 2/10/01, 9/27/00

The Regional Planning Commission, developer, and engineer are advised that:

PRIOR TO RECOMMENDING APPROVAL OF THE TENTATIVE TRACT OR PARCEL MAP:

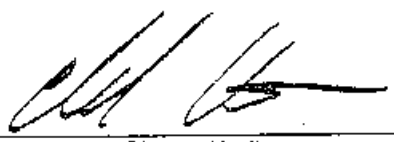
1. The base used for the geologic map does not conform to the most recent tentative map submitted through the Department of Regional Planning. This review is based solely on the geologic map, and additional review comments may be provided after receipt of a revised tentative map.

2. Show proposed final topography for landslides to be completely or partially removed.

3. The proposed cut slope on sheet 11 above trench T-23L may expose a daylighted component of bedding. Provide a geologic cross section(s) and stability analyses for this slope, and any mitigation recommendations as necessary. Note: grading plan review will require continuous stratigraphy be provided to further address stability of this cut.

4. The Soils Engineering review dated 2/5/10 is attached.

NOTE Provide a copy of this review with your resubmittal

Prepared by  Reviewed by _____ Date 2/5/10

Charles Nestle

Please complete a Customer Service Survey at http://dpw.lacounty.gov/go/gmedsurvey

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL AND MATERIALS ENGINEERING DIVISION

SOILS ENGINEERING REVIEW SHEET

Address: 900 S. Fremont Ave., Alhambra, CA 91803
Telephone: (626) 458-4925
Fax: (626) 458-4913

District Office 8.2
Job Number GMPH
Sheet 1 of 1

DISTRIBUTION:
___ Drainage
___ Grading
___ Geo/Soils Central File
___ District Engineer
___ Geologist
___ Soils Engineer
___ Engineer/Architect

Tentative Tract Map	53108
Location	Newhall Ranch
Developer/Owner	Newhall Land and Farming Company / Newhall Ranch Co.
Engineer/Architect	Psomas
Soils Engineer	Allan E. Seward Engineering Geology, Inc.
Geologist	Same as above

Review of:
Revised Tentative Tract Map and Exhibit Dated 5/2/07
Soils Engineering and Geologic Reprot Dated 12/21/09, 10/29/03, 2/10/01, 9/27/00
Previous Review Sheet Dated 11/13/08

ACTION:

Tentative Map feasibility is not recommenced for approval.

REMARKS:

1. The geotechnical map within the submitted report does not match the latest approved Tentative Tract Map dated by Regional Planning 5/2/07. Provide revised Tentative Tract Map dated by Regional Planning and geotechnical map that conform.

2. Show the final graded contours of all landslides to be removed and recompacted on the geotechnical maps and Tentative Map. For example, Landslide Qls XVI(B) is recommended to removed and recompacted, however the final graded contours are not shown on the geotechnical map or the Tentative Map. Revise geotechnical maps and Tentative Map as necessary.

3. On the geotechnical map, delineate approximate limits of proposed disposal fills. Also delineate all soils unsuitable for the support of structures (proposed disposal fill, landslide debris) and prominently note/label with the note: "Unsuitable Soils - Not suitable for the Support of Structures".

4. Discuss and address whether the proposed fill slopes associated with Disposal Fill areas will be subject to erosion and/or slope instability. Provide updated surficial, static, and seismic slope stability analyses considering the density, moisture content, and shear strengths of the disposal fill materials as well as the proposed design shown on the latest geotechnical map and Tentative Map. Provide additional recommendations as necessary.

5. Proposed Disposal Fill areas DF-1 and DF-2 appear to show debris basins located in disposal fill. Clarify the soil material that will be used to construct the proposed debris basins and provide recommendations as necessary.

6. Provice static and seismic slope stability analyses for the proposed cut slopes surrounding the Zone 1A Water Tank at PM 18108. The referenced R.T. Frankian Report dated 3/28/08 does not include slope stability analyses. Also, provide a geotechnical cross section, for each section analyzed, showing the critical failure plane used in the analyses. Indicate the various shear strength parameters used in the analyses, in the appropriate segments of each failure plane. Show locations of the cross sections used in slope stability analyses on the geotechnical map. Recommend mitigation if factors of safety are below County minimum standards.

7. The submitted report states that the design of the bank protection has been modified. Provide clarification, additional discussion, and details in regards to the stated design modification in relation to proposed bank protection shown on both the North and South banks of the Santa Clara River. The proposed Bank Stabilization is necessary for the feasibility of the proposed tract development and therefore must be addressed at the Tentative Map stage. Clarify design and provide additional information as necessary.

8. Show the following on the geotechnical map and Tentative Map:
   a. Final graded contours for the removal and recompaction of existing landslides.
   b. Delineate the location of the proposed disposal fill materials.
   c. Delineate the approximate limits of all soils unsuitable for the support of structure and prominently note/label with the note: "Unsuitable Soils - Not suitable for the Support of Structures".
   d. All recommended mitigation measures.

9. Requirements of the Geology Section are attached.

10. Include a copy of this review sheet with your response.

REGISTERED PROFESSIONAL ENGINEER
YOSHIYA MORISAKU
No. 67587
Exp. 6/30/11
CIVIL
STATE OF CALIFORNIA

Reviewed by _____ Date 2/5/09
Yoshiya Morisaku

NOTICE: Public safety, relative to geotechnical subsurface explorations, is provided in accordance with current codes for excavations, including of the Los Angeles County Code, Chapter 11.48, and the State of California, Title 8, Construction Safety Orders.

Appendix A

APPENDIX A

SLOPE STABILITY ANALYSES

Introduction

Analyses were performed on Cross Sections 2A-2A' and 3A-3A' to evaluate stability of proposed fill and natural slopes in offsite areas north of Highway 126. Analyses were also performed on Cross Section 4A-4A' to evaluate stability of soil-cement channel liner slopes, considering potential rapid drawdown conditions and utilizing the Qcap flood event water level and assuming that the aesthetic 3:1 fill slope in front of the liner has been removed by erosion. The stability analyses utilized cross sections constructed to illustrate critical structural geometries and maximum slope heights for each analyzed slope. Our evaluation included static and pseudostatic gross stability conditions. The results of our stability analyses are summarized in **Table A1** along with slope parameters and recommended mitigations. Slope stability diagrams graphically illustrating the results of our analyses for each cross section are attached for review.

Shear Strength Parameters

Shear strength parameters used in our analyses for the slopes within and adjacent to the proposed fill disposal site areas north of Highway 126 are based on direct shear strength testing performed for the Homestead Project, Vesting Tentative Tract 060678. The shear strength assigned for artificial fill and proposed uncompacted fill was reduced 20% from compacted fill for purposes of analysis. Shear strength parameters used to evaluate channel liner slopes were developed for our original Review of Tentative Tract Map report for VTT 53108 (dated September 27, 2000). Shear strength parameters of phi=45 degrees and cohesion=2000 psf were selected for soil cement. These values are conservative based on unconfined compressive strength tests performed on soil-cement cylinders prepared for similar projects. The strength parameters for static loading conditions are residual values and the strength values for pseudo-static loading conditions are peak values.

Conclusion

The results of our stability analyses are presented in **Table A1 -** Slope Stability Analyses Results. General descriptions of geologic/geotechnical conditions at the site are provided in the referenced reports.

APPENDIX A

The analyzed slopes comply with County of Los Angeles Building Code minimum requirements for global stability under static, pseudostatic, and rapid drawdown loading conditions, as applicable, provided that our recommendations are followed and incorporated into project construction.

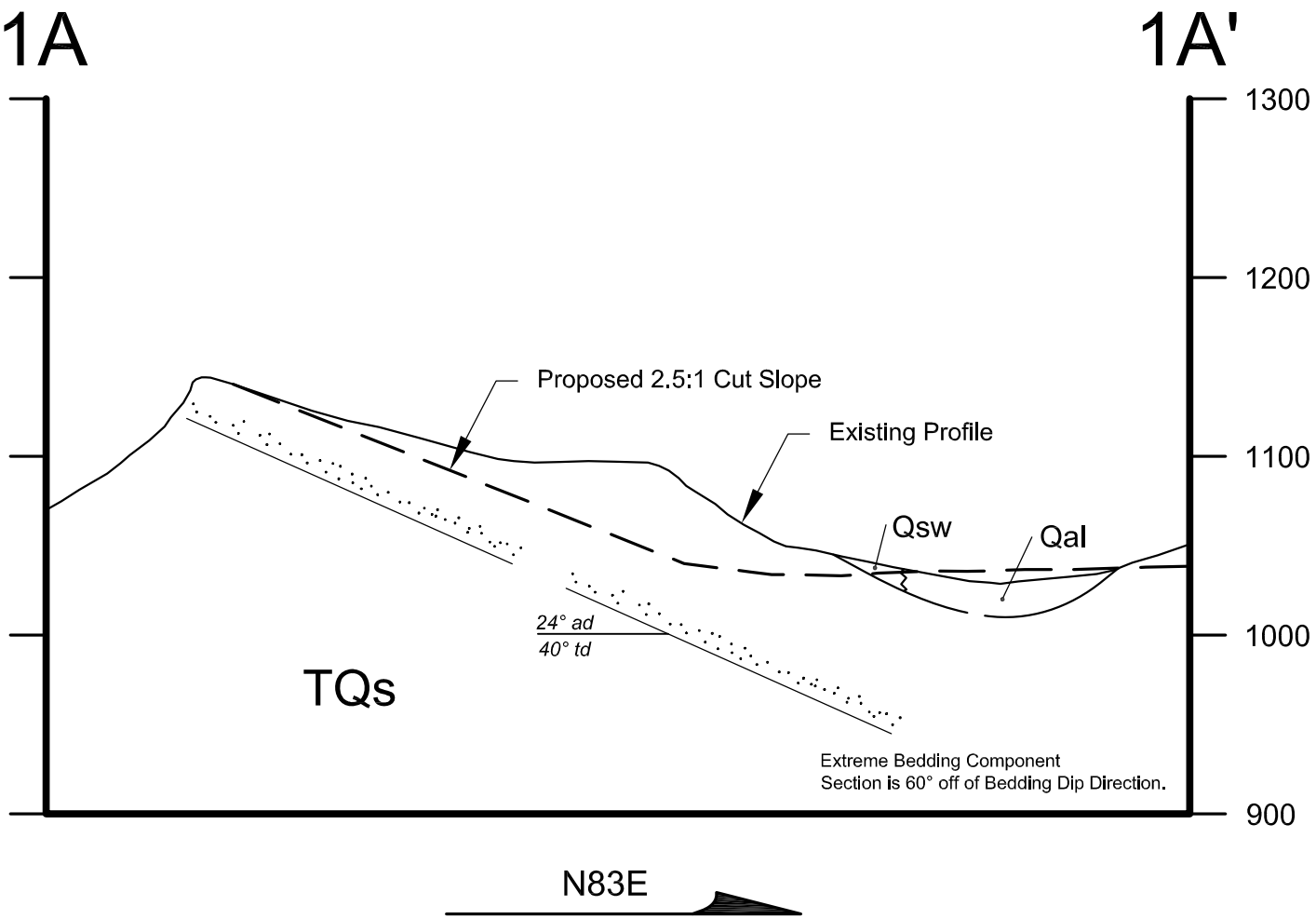
The following attachments are located within this Appendix.

Slope Stability Analyses Results **Table A1**
Slope Stability Diagrams and Data Sheets for Runs 1 through 7

APPENDIX A

TABLE A1 - SLOPE STABILITY ANALYSES RESULTS

CROSS SECTION	FILE NAME	SLOPE DESIGNATION	ANALYZED SURFACE	METHOD	FACTOR OF SAFETY		COMMENTS	RUN NO.
					STATIC	PSEUDOSTATIC (COEFF = 0.15)		
2A-2A'	T2AS2A	Natural slope w/ surficial failure (Qls-XVIB)	Landslide Plane	Spencer	2.77	--	Minimum safety factor for potential failure surface along landslide plane located behind proposed compacted disposal fill.	1
2A-2A'	T2AS1A T2APS1A	Natural Slope	Bedding Plane	Spencer	3.10	1.87	Minimum safety factor for potential failure along bedding plane located below proposed compacted disposal fill.	2 3
3A-3A'	T3AS1A T3APS1A	Fill Slope	Circular	Spencer	1.74	1.18	Circular analyses through fill slope below proposed basin; Ru=0.12.	4 5
4A-4A'	T4AS1A T4APS1A	Soil-Cement Bank Protection	Circular	Spencer	2.24	1.56	Minimum safety factor considering potential rapid drawdown. Water elevation in slope material equal to Qcap.	6 7

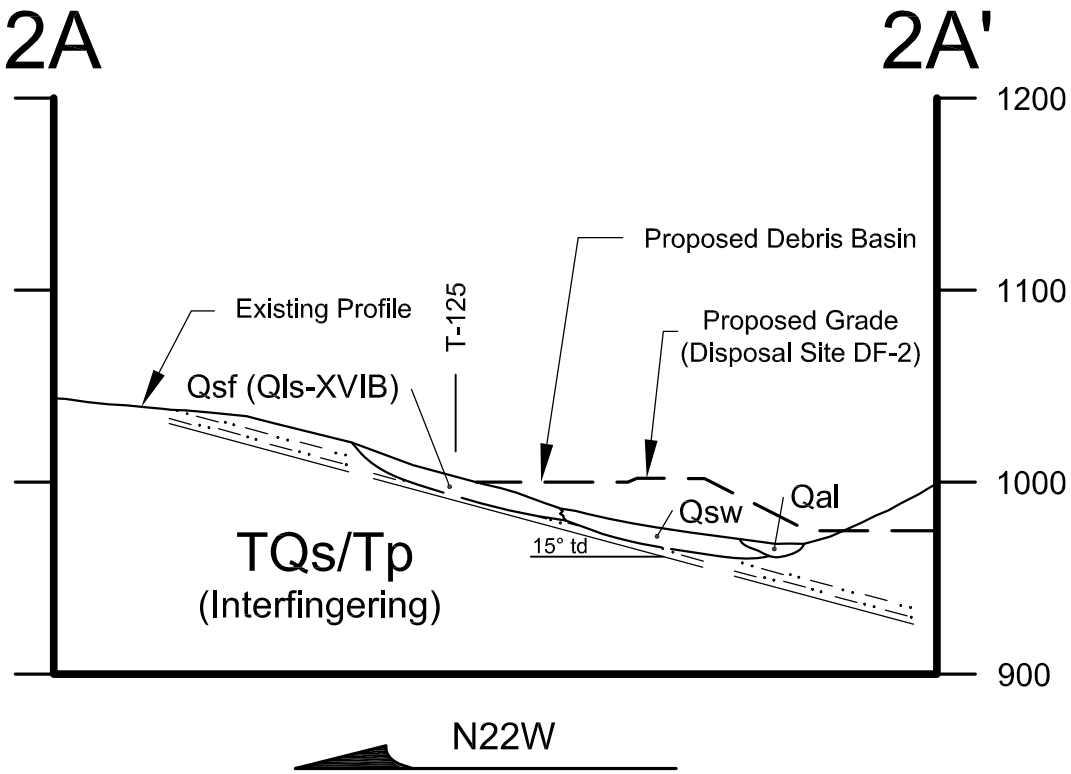


ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

Cross Section 1A-1A '

Job No.: 10-1702R

Date: 2/25/10 | *Scale:* 1"=100' (H=V)

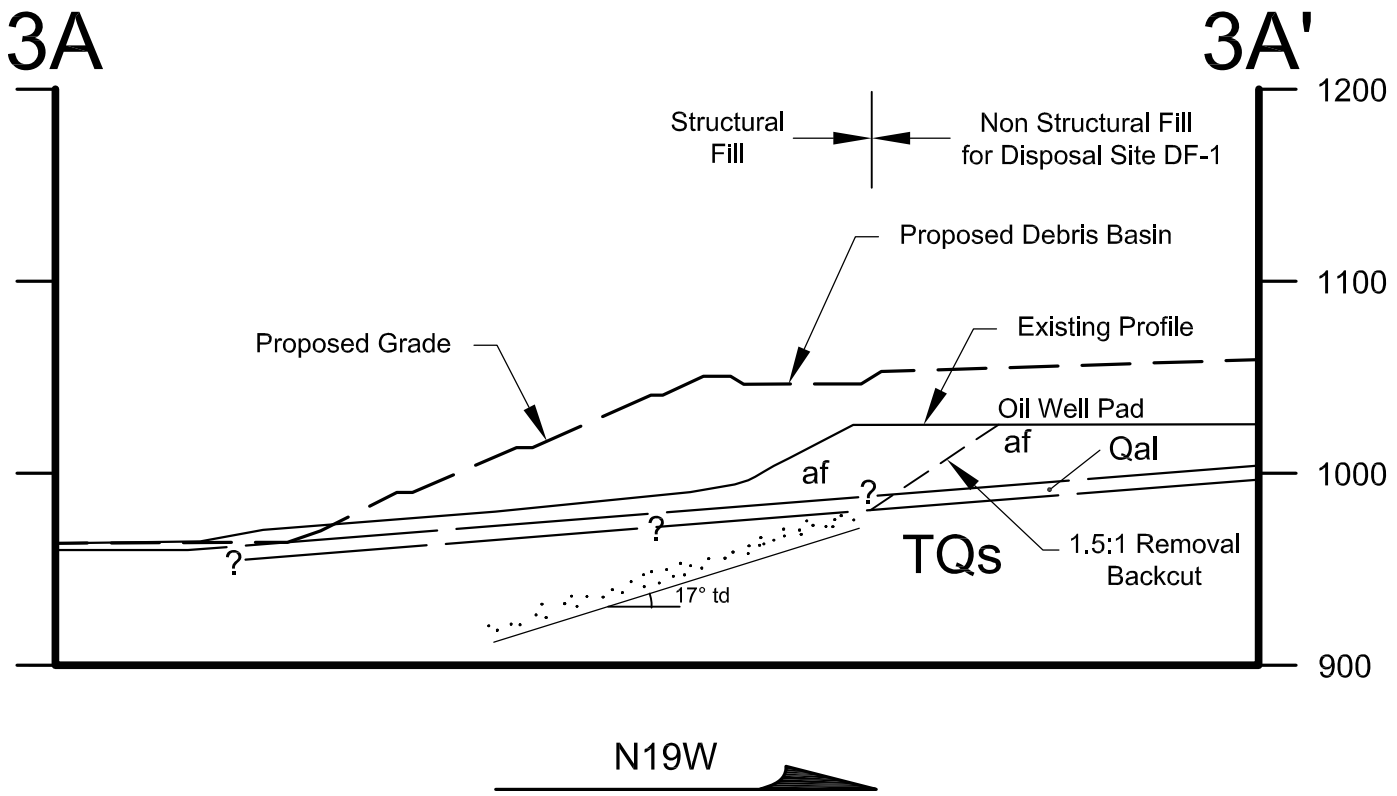


ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

Cross Section 2A-2A'

Job No.: 10-1702R

Date: 2/25/10 | *Scale:* 1"=100' (H=V)

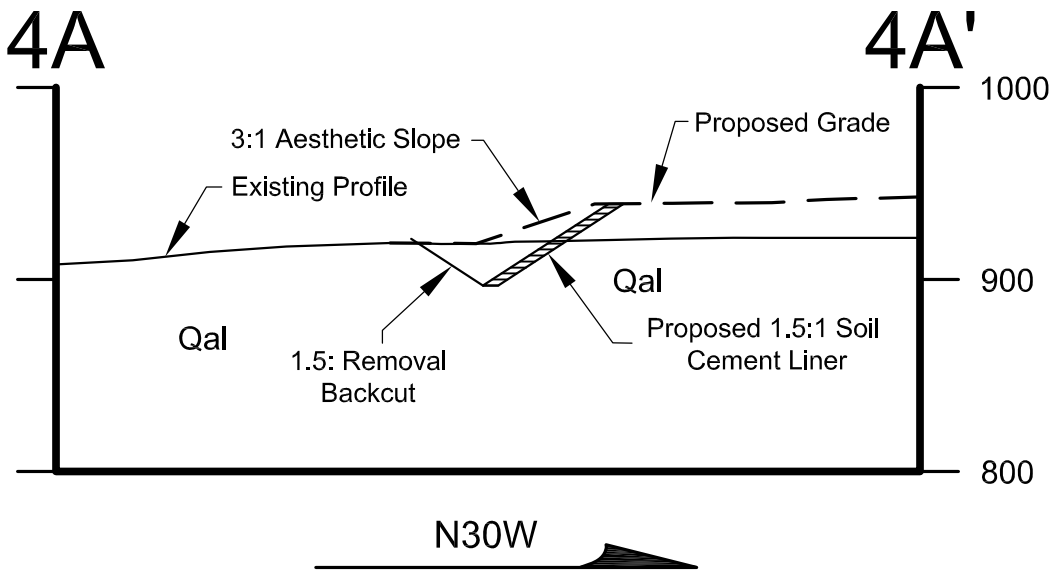


ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

Cross Section 3A-3A'

Job No.: 10-1702R

Date: 2/25/10 | *Scale:* 1"=100' (H=V)



ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

Cross Section 4A-4A'

Job No.: 10-1702R

Date: 2/25/09 | *Scale:* 1"=100' (H=V)

TT 53108, 2A-2A'; QlsXVIB; static; slide plane

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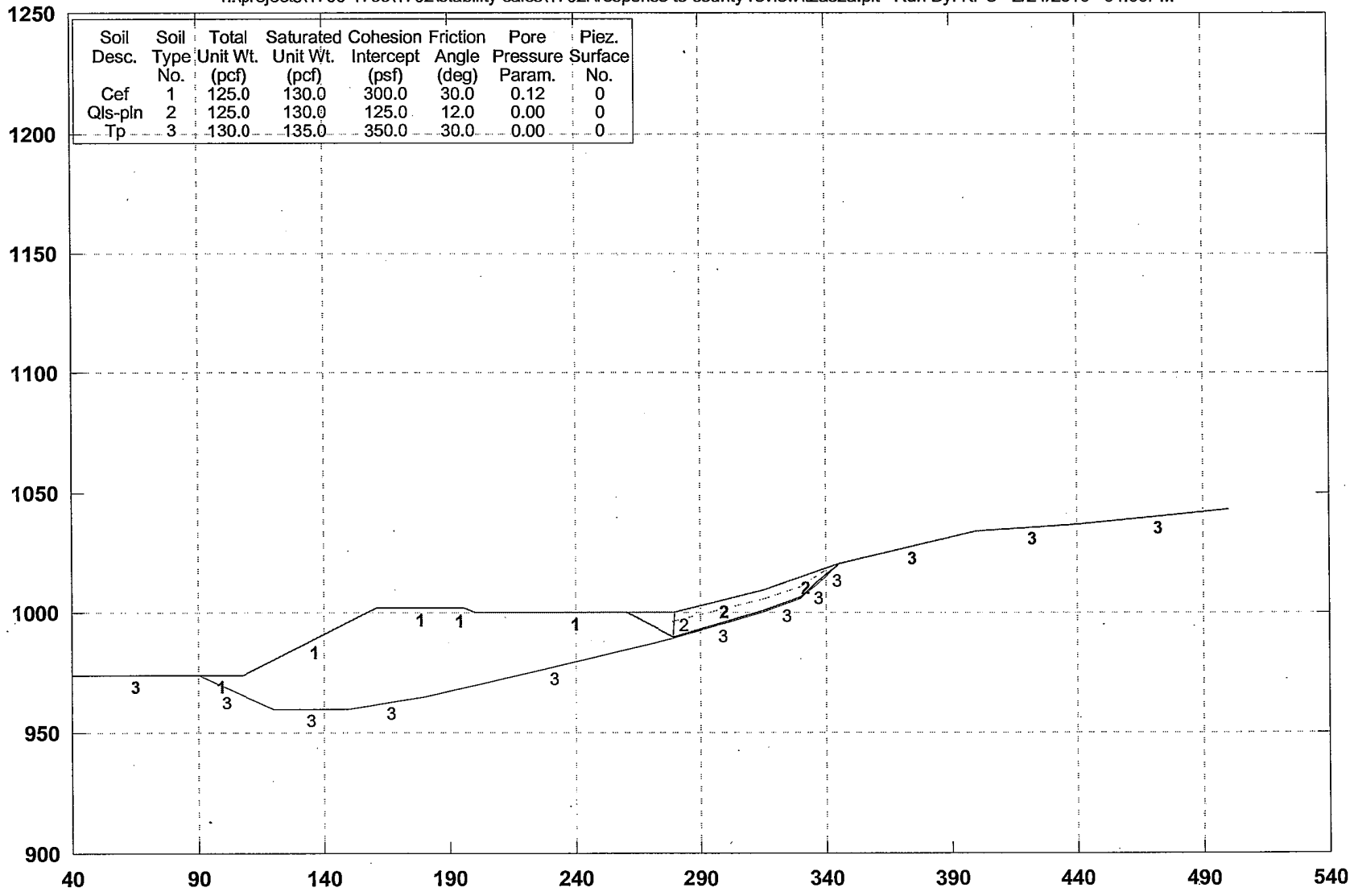
Cef	1	125.0	130.0	300.0	30.0	0.12	0
Qls-pln	2	125.0	130.0	125.0	12.0	0.00	0
Tp	3	130.0	135.0	350.0	30.0	0.00	0



GSTABL7 v.2 FSmin=2.77

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

RUN No. 1

```
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                ** GSTABL7 by Garry H. Gregory, P.E. **
    ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
              (All Rights Reserved-Unauthorized Use Prohibited)
 ************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
 ************************************************************************
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  Time of Run:              04:00PM
  Run By:                   KPC
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  Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t2as2a.OUT
  Unit System:              English
  Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t2as2a.PLT
  PROBLEM DESCRIPTION:  TT 53108, 2A-2A'; QlsXVIB;
                        static; slide plane
  BOUNDARY COORDINATES
    11 Top   Boundaries
    20 Total Boundaries
  Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
     No.        (ft)        (ft)       (ft)        (ft)     Below Bnd
      1         40.00      974.00      90.00      974.00        3
      2         90.00      974.00     108.00      974.00        1
      3        108.00      974.00     161.00     1002.00        1
      4        161.00     1002.00     196.00     1002.00        1
      5        196.00     1002.00     200.00     1000.00        1
      6        200.00     1000.00     280.00     1000.00        1
      7        280.00     1000.00     315.00     1009.00        2
      8        315.00     1009.00     345.00     1020.00        2
      9        345.00     1020.00     400.00     1034.00        3
     10        400.00     1034.00     440.00     1037.00        3
     11        440.00     1037.00     500.00     1043.00        3
     12         90.00      974.00     120.00      960.00        3
     13        120.00      960.00     150.00      960.00        3
     14        150.00      960.00     180.00      965.00        3
     15        180.00      965.00     279.00      989.00        3
     16        279.00      989.00     280.00     1000.00        2
     17        279.00      989.00     315.00     1000.00        3
     18        315.00     1000.00     330.00     1006.00        3
     19        330.00     1006.00     340.00     1015.00        3
     20        340.00     1015.00     345.00     1020.00        3
  User Specified Y-Origin =       900.00(ft)
  Default X-Plus Value = 0.00(ft)
  Default Y-Plus Value = 0.00(ft)
 ISOTROPIC SOIL PARAMETERS
   3 Type(s) of Soil
  Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
  Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
   No.  (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
    1   125.0    130.0     300.0    30.0    0.12       0.0       0
    2   125.0    130.0     125.0    12.0    0.00       0.0       0
    3   130.0    135.0     350.0    30.0    0.00       0.0       0
  Trial Failure Surface Specified By  6 Coordinate Points
    Point      X-Surf      Y-Surf
     No.        (ft)        (ft)
      1        260.00     1000.00
      2        279.00      989.50
      3        315.00     1000.50
```

4	330.00	1006.50
5	339.50	1015.50
6	344.50	1019.82

Circle Center At X = 286.814(ft) ; Y = 1075.730(ft); and Radius = 80.337(ft)

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
13.00	2.328	3.037	0.231
19.50	0.000	3.606	0.354
10.88	2.570	2.893	0.192
6.87	2.907	2.660	0.121
7.85	2.838	2.713	0.138
9.17	2.731	2.789	0.161
8.61	2.778	2.756	0.151
8.76	2.765	2.765	0.154

((Modified Bishop FS for Specified Surface = 2.328))
Factor Of Safety For The Preceding Specified Surface = 2.765
Theta (ki = 1.0) = 8.76 Deg Lambda = 0.154
Factor Of Safety Is Calculated By GLE (Spencer's) Method (0-2)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails
(if applicable) have been applied to the slice base(s)
on which they intersect.
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
Tension Crack Water Force = 0.00(lbs)
Specified Tension Crack Water Force Factor = 0.000
Depth of Tension Crack (zo) at Center of Last Slice = 0.777(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	279.00	996.21	0.639	15532.	1.000	8.76	2394.6
2	279.05	996.22	0.640	15530.	1.000	8.76	2394.3
3	280.00	996.47	0.653	15306.	1.000	8.76	2359.7
4	315.00	1005.51	0.589	8023.	1.000	8.76	1236.8
5	330.00	1010.67	0.521	4093.	0.858	7.53	464.5
6	339.50	1017.41	0.769	195.	0.296	2.61	2.6
7	344.50	1058.07	0.000	-1.	0.000	0.00	0.0

Table 1 - Individual data on the 7 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	19.0	12468.8	0.0	1709.5	0.0	0.0	0.0	0.0	0.0
2	0.0	61.3	0.0	7.7	0.0	0.0	0.0	0.0	0.0
3	1.0	1232.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	35.0	40894.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	15.0	15468.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	9.5	6224.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	5.0	776.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the 7 Slices -

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	-28.93	269.50	21.71	0.00	574.38	0.00
2	16.99	279.02	0.05	1205.28	1254.36	327.31
3	16.99	279.52	1.00	1214.33	1236.09	138.56
4	16.99	297.50	36.60	1097.11	1117.40	129.55
5	21.80	322.50	16.16	938.59	957.49	117.36
6	43.45	334.75	13.09	519.01	475.65	85.11
7	40.81	342.00	6.61	107.28	117.49	53.46

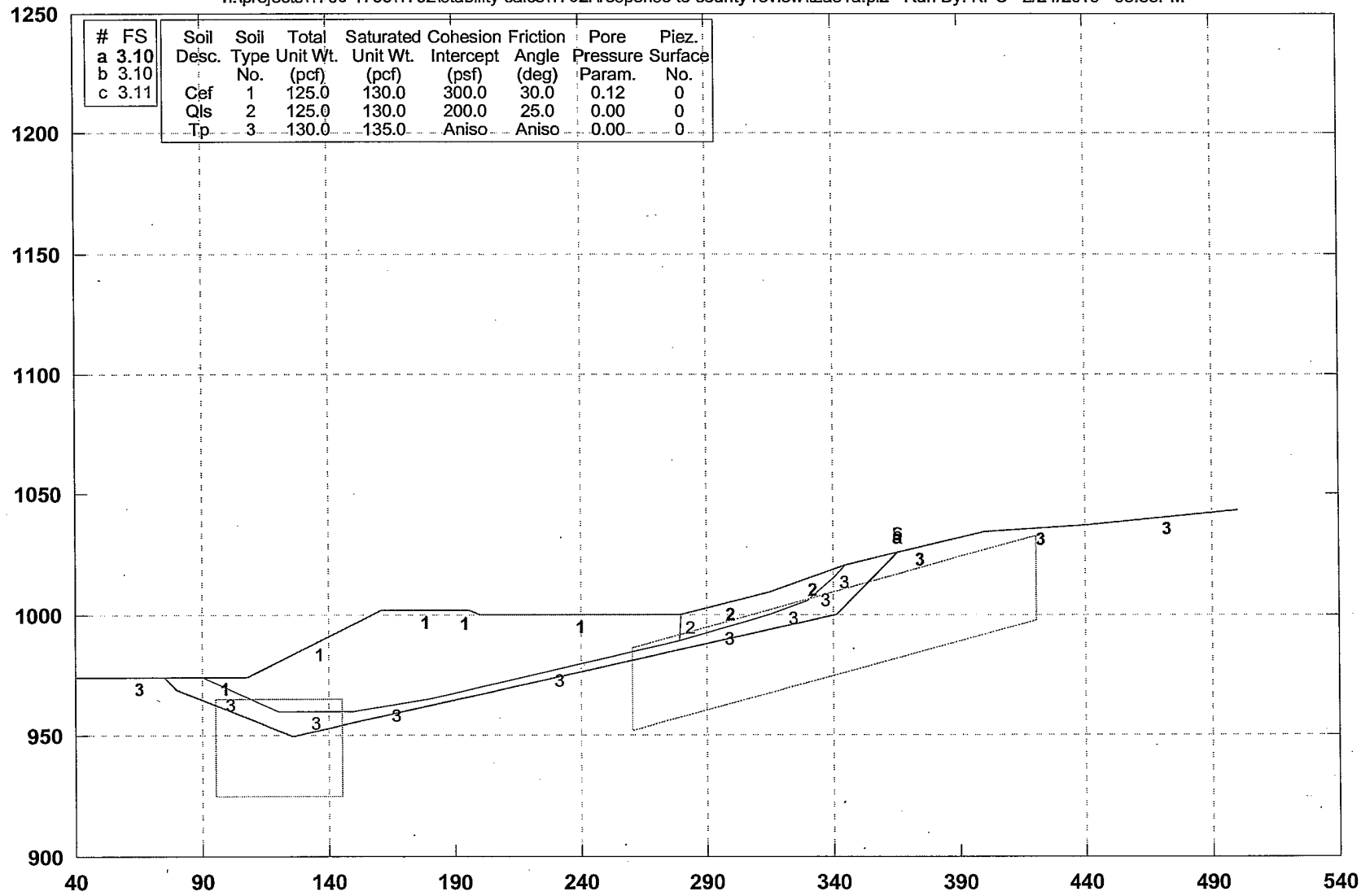
**** END OF GSTABL7 OUTPUT ****

TT 53108, 2A-2A'; QlsXVIB; static; planar below R&R

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t2as1a.pl2 Run By: KPC 2/24/2010 03:53PM

#	FS
a	3.10
b	3.10
c	3.11

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0.12	0
Qls	2	125.0	130.0	200.0	25.0	0.00	0
Tp	3	130.0	135.0	Aniso	Aniso	0.00	0



GSTABL7 v.2 FSmin=3.10

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

RUN NO. 2

```
                              *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
          ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                    (All Rights Reserved-Unauthorized Use Prohibited)
     ************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
               Modified Bishop, Simplified Janbu, or GLE Method of Slices.
               (Includes Spencer & Morgenstern-Price Type Analysis)
               Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
               Nonlinear Undrained Shear Strength, Curved Phi Envelope,
               Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
               Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
     ************************************************************************
     Analysis Run Date:        2/24/2010
     Time of Run:              03:53PM
     Run By:                   KPC
     Input Data Filename:      n:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t2as1a.in
     Output Filename:          n:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t2as1a.OUT
     Unit System:              English
     Plotted Output Filename:  n:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t2as1a.PLT
     PROBLEM DESCRIPTION:  TT 53108, 2A-2A'; QlsXVIB;
                           static; planar below R&R
     BOUNDARY COORDINATES
        11 Top   Boundaries
        20 Total Boundaries
     Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
        No.        (ft)       (ft)       (ft)       (ft)    Below Bnd
         1        40.00     974.00      90.00     974.00        3
         2        90.00     974.00     108.00     974.00        1
         3       108.00     974.00     161.00    1002.00        1
         4       161.00    1002.00     196.00    1002.00        1
         5       196.00    1002.00     200.00    1000.00        1
         6       200.00    1000.00     280.00    1000.00        1
         7       280.00    1000.00     315.00    1009.00        2
         8       315.00    1009.00     345.00    1020.00        2
         9       345.00    1020.00     400.00    1034.00        3
        10       400.00    1034.00     440.00    1037.00        3
        11       440.00    1037.00     500.00    1043.00        3
        12        90.00     974.00     120.00     960.00        3
        13       120.00     960.00     150.00     960.00        3
        14       150.00     960.00     180.00     965.00        3
        15       180.00     965.00     279.00     989.00        3
        16       279.00     989.00     280.00    1000.00        2
        17       279.00     989.00     315.00    1000.00        3
        18       315.00    1000.00     330.00    1006.00        3
        19       330.00    1006.00     340.00    1015.00        3
        20       340.00    1015.00     345.00    1020.00        3
     User Specified Y-Origin =        900.00(ft)
     Default X-Plus Value = 0.00(ft)
     Default Y-Plus Value = 0.00(ft)
    ISOTROPIC SOIL PARAMETERS
      3 Type(s) of Soil
     Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
     Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
      No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
       1   125.0    130.0     300.0     30.0    0.12       0.0       0
       2   125.0    130.0     200.0     25.0    0.00       0.0       0
       3   130.0    135.0     350.0     30.0    0.00       0.0       0
    ANISOTROPIC STRENGTH PARAMETERS
         1 soil type(s)
     Soil Type  3 Is Anisotropic
     Number Of Direction Ranges Specified =  3
     Direction     Counterclockwise     Cohesion      Friction
       Range        Direction Limit    Intercept       Angle
```

No.	(deg)	(psf)	(deg)
1	13.0	350.00	30.00
2	17.0	200.00	20.00
3	90.0	350.00	30.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
2500 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	95.00	945.00	145.00	945.00	40.00
2	260.00	969.00	420.00	1015.00	35.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 516
Number of Trial Surfaces with Misleading FS = 15
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 21.2 %
Statistical Data On All Valid FS Values:
FS Max = 24.684 FS Min = 3.096 FS Ave = 4.874
Standard Deviation = 1.493 Coefficient of Variation = 30.63 %
((Simplified Janbu FS for Critical Surface = 2.785))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	74.90	974.00
2	79.72	969.17
3	125.62	949.35
4	341.04	999.95
5	365.66	1025.26

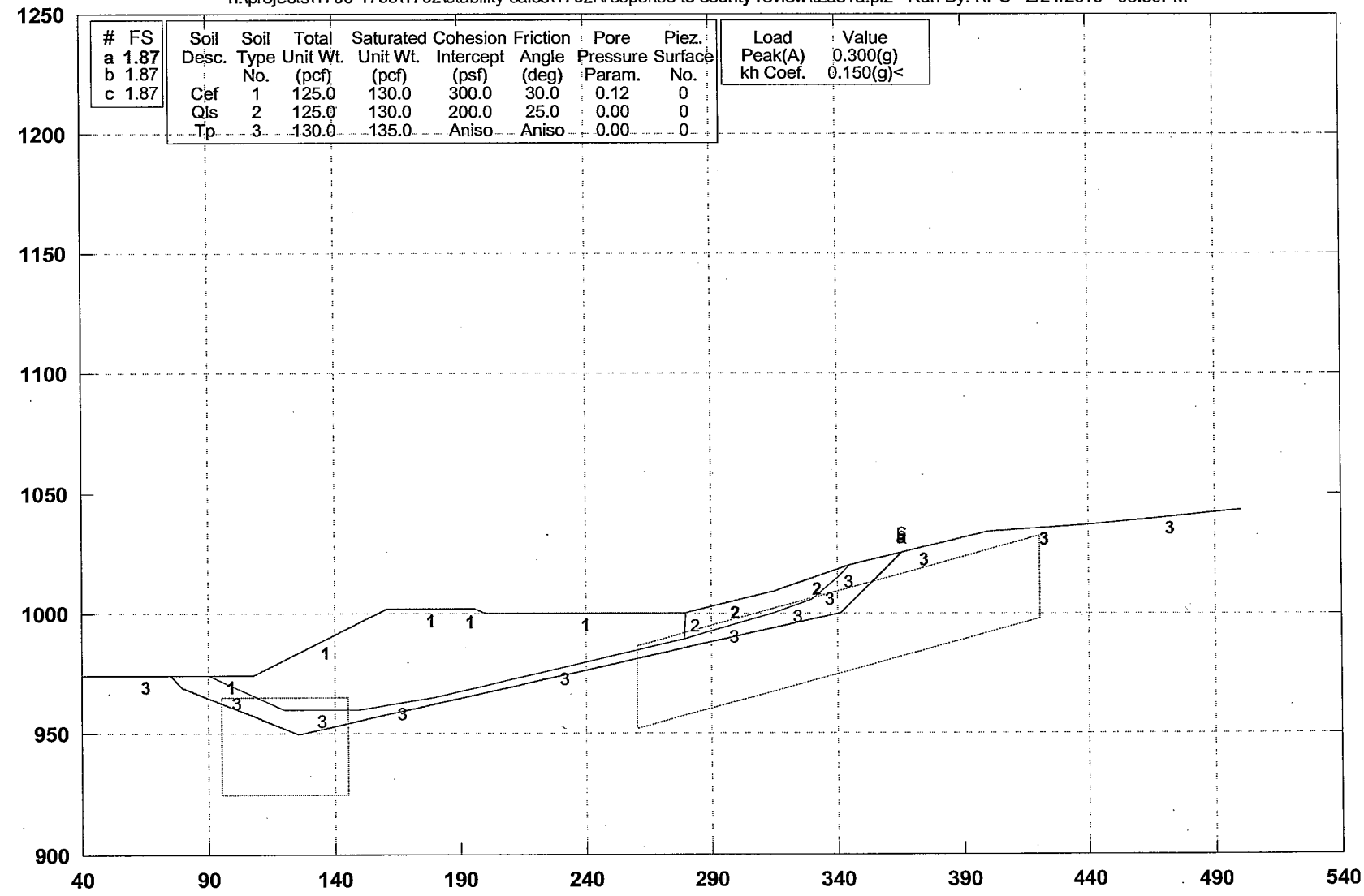
*** FOS = 3.096 Theta (ki=1.0) = 7.97 ***
Lambda = 0.140
Individual data on the 18 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.8	1513.8	0.0	0.0	0.	0.	0.0	0.0	0.0
2	10.3	9413.9	0.0	0.0	0.	0.	0.0	0.0	0.0
3	18.0	30394.1	0.0	0.0	0.	0.	0.0	0.0	0.0
4	12.0	34701.6	0.0	0.0	0.	0.	0.0	0.0	0.0
5	5.6	22241.2	0.0	0.0	0.	0.	0.0	0.0	0.0
6	24.4	115331.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	11.0	59015.4	0.0	0.0	0.	0.	0.0	0.0	0.0

8	19.0	100388.8	0.0	0.0	0.	0.	0.0	0.0	0.0
9	16.0	76286.8	0.0	0.0	0.	0.	0.0	0.0	0.0
10	4.0	17401.2	0.0	0.0	0.	0.	0.0	0.0	0.0
11	79.0	237861.8	0.0	0.0	0.	0.	0.0	0.0	0.0
12	1.0	1840.6	0.0	0.0	0.	0.	0.0	0.0	0.0
13	35.0	65870.8	0.0	0.0	0.	0.	0.0	0.0	0.0
14	15.0	31022.1	0.0	0.0	0.	0.	0.0	0.0	0.0
15	10.0	22976.2	0.0	0.0	0.	0.	0.0	0.0	0.0
16	1.0	2515.7	0.0	0.0	0.	0.	0.0	0.0	0.0
17	4.0	8912.1	0.0	0.0	0.	0.	0.0	0.0	0.0
18	20.7	21569.1	0.0	0.0	0.	0.	0.0	0.0	0.0

TT 53108, 2A-2A'; QlsXVIB; pseudo; planar below R&R

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t2as1a.pl2 Run By: KPC 2/24/2010 03:59PM



#	FS
a	1.87
b	1.87
c	1.87

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0.12	0
Qls	2	125.0	130.0	200.0	25.0	0.00	0
Tp	3	130.0	135.0	Aniso	Aniso	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.87

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

RUN NO. 3

```
                    *** GSTABL7 ***
               ** GSTABL7 by Garry H. Gregory, P.E. **
   ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
             (All Rights Reserved-Unauthorized Use Prohibited)
 ************************************************************************
                  SLOPE STABILITY ANALYSIS SYSTEM
      Modified Bishop, Simplified Janbu, or GLE Method of Slices.
      (Includes Spencer & Morgenstern-Price Type Analysis)
      Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
      Nonlinear Undrained Shear Strength, Curved Phi Envelope,
      Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
      Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
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  Time of Run:              03:59PM
  Run By:                   KPC
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County Review\t2as1a.in
  Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t2as1a.OUT
  Unit System:              English
  Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t2as1a.PLT
  PROBLEM DESCRIPTION:  TT 53108, 2A-2A'; QlsXVIB;
                        pseudo; planar below R&R
  BOUNDARY COORDINATES
     11 Top   Boundaries
     20 Total Boundaries
  Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
     No.        (ft)        (ft)       (ft)        (ft)      Below Bnd
      1         40.00      974.00      90.00      974.00        3
      2         90.00      974.00     108.00      974.00        1
      3        108.00      974.00     161.00     1002.00        1
      4        161.00     1002.00     196.00     1002.00        1
      5        196.00     1002.00     200.00     1000.00        1
      6        200.00     1000.00     280.00     1000.00        1
      7        280.00     1000.00     315.00     1009.00        2
      8        315.00     1009.00     345.00     1020.00        2
      9        345.00     1020.00     400.00     1034.00        3
     10        400.00     1034.00     440.00     1037.00        3
     11        440.00     1037.00     500.00     1043.00        3
     12         90.00      974.00     120.00      960.00        3
     13        120.00      960.00     150.00      960.00        3
     14        150.00      960.00     180.00      965.00        3
     15        180.00      965.00     279.00      989.00        3
     16        279.00      989.00     280.00     1000.00        2
     17        279.00      989.00     315.00     1000.00        3
     18        315.00     1000.00     330.00     1006.00        3
     19        330.00     1006.00     340.00     1015.00        3
     20        340.00     1015.00     345.00     1020.00        3
  User Specified Y-Origin =       900.00(ft)
  Default X-Plus Value = 0.00(ft)
  Default Y-Plus Value = 0.00(ft)
 ISOTROPIC SOIL PARAMETERS
   3 Type(s) of Soil
  Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
  Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
   No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
    1   125.0    130.0     300.0     30.0    0.12      0.0       0
    2   125.0    130.0     200.0     25.0    0.00      0.0       0
    3   130.0    135.0     500.0     36.0    0.00      0.0       0
 ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
  Soil Type  3 Is Anisotropic
  Number Of Direction Ranges Specified =  3
  Direction     Counterclockwise     Cohesion      Friction
    Range       Direction Limit     Intercept       Angle
```

No.	(deg)	(psf)	(deg)
1	13.0	500.00	36.00
2	17.0	200.00	20.00
3	90.0	500.00	36.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Specified Peak Ground Acceleration Coefficient (A) = 0.300(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.
2500 Trial Surfaces Have Been Generated.
2 Boxes Specified For Generation Of Central Block Base
Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 50.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	95.00	945.00	145.00	945.00	40.00
2	260.00	969.00	420.00	1015.00	35.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 323
Number of Trial Surfaces with Misleading FS = 18
Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Evaluated = 13.6 %
Statistical Data On All Valid FS Values:
FS Max = 4.667 FS Min = 1.865 FS Ave = 2.910
Standard Deviation = 0.578 Coefficient of Variation = 19.86 %
((Simplified Janbu FS for Critical Surface = 1.697))
Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	74.90	974.00
2	79.72	969.17
3	125.62	949.35
4	341.04	999.85
5	365.66	1025.26

*** FOS = 1.865 Theta (ki=1.0) = 6.76 ***
Lambda = 0.119
Individual data on the 18 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.8	1513.8	0.0	0.0	0.	0.	227.1	0.0	0.0
2	10.3	9413.9	0.0	0.0	0.	0.	1412.1	0.0	0.0
3	18.0	30394.1	0.0	0.0	0.	0.	4559.1	0.0	0.0

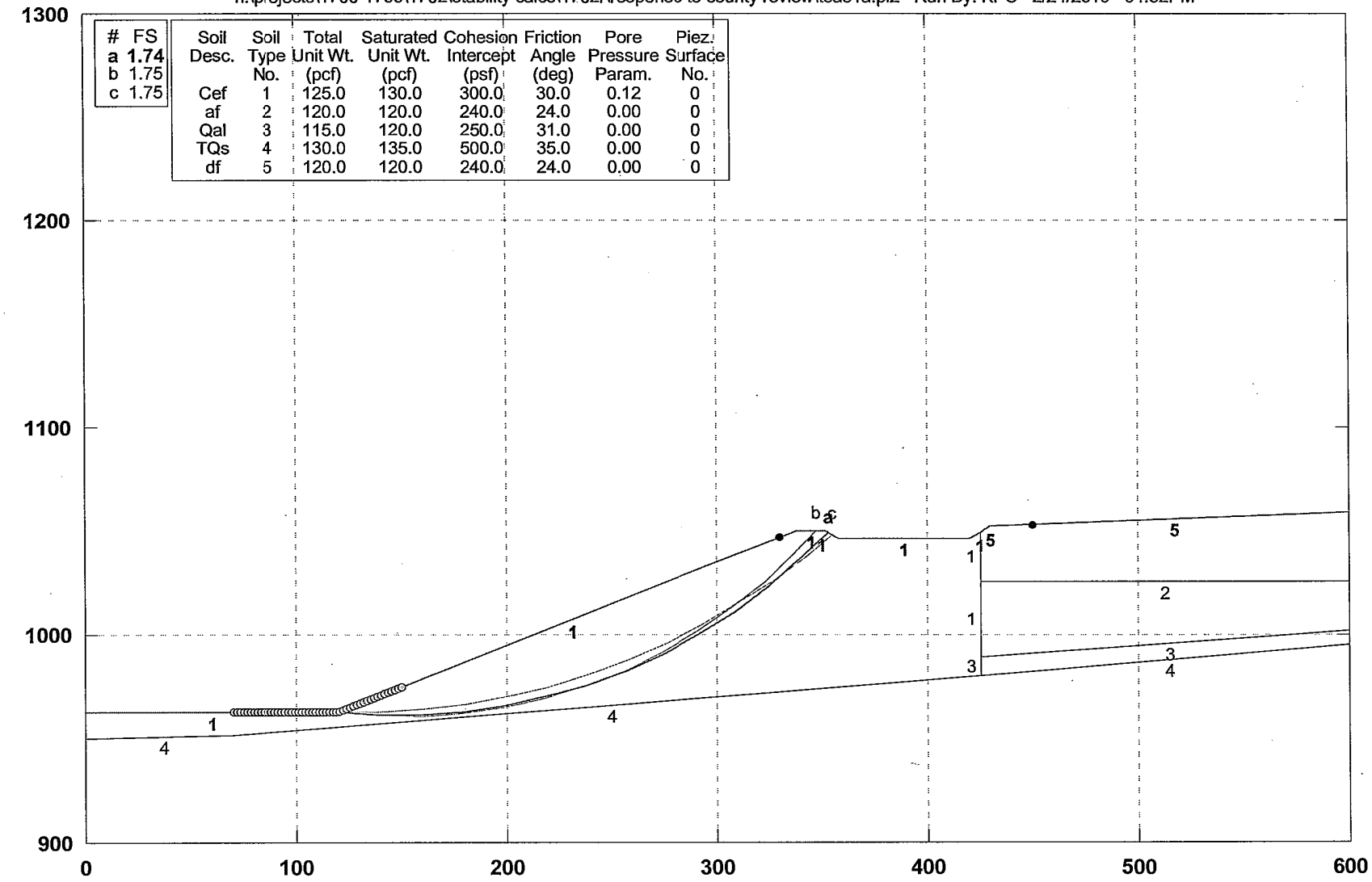
4	12.0	34701.6	0.0	0.0	0.	0.	5205.2	0.0	0.0
5	5.6	22241.2	0.0	0.0	0.	0.	3336.2	0.0	0.0
6	24.4	115331.2	0.0	0.0	0.	0.	17299.7	0.0	0.0
7	11.0	59015.4	0.0	0.0	0.	0.	8852.3	0.0	0.0
8	19.0	100388.8	0.0	0.0	0.	0.	15058.3	0.0	0.0
9	16.0	76286.8	0.0	0.0	0.	0.	11443.0	0.0	0.0
10	4.0	17401.2	0.0	0.0	0.	0.	2610.2	0.0	0.0
11	79.0	237861.8	0.0	0.0	0.	0.	35679.3	0.0	0.0
12	1.0	1840.6	0.0	0.0	0.	0.	276.1	0.0	0.0
13	35.0	65870.8	0.0	0.0	0.	0.	9880.6	0.0	0.0
14	15.0	31022.1	0.0	0.0	0.	0.	4653.3	0.0	0.0
15	10.0	22976.2	0.0	0.0	0.	0.	3446.4	0.0	0.0
16	1.0	2515.7	0.0	0.0	0.	0.	377.4	0.0	0.0
17	4.0	8912.1	0.0	0.0	0.	0.	1336.8	0.0	0.0
18	20.7	21569.1	0.0	0.0	0.	0.	3235.4	0.0	0.0

TT 53108, 3A-3A'; fill slope; static; circular

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t3as1a.pl2 Run By: KPC 2/24/2010 04:52PM

#	FS
a	1.74
b	1.75
c	1.75

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0.12	0
af	2	120.0	120.0	240.0	24.0	0.00	0
Qal	3	115.0	120.0	250.0	31.0	0.00	0
TQs	4	130.0	135.0	500.0	35.0	0.00	0
df	5	120.0	120.0	240.0	24.0	0.00	0



GSTABL7 v.2 FSmin=1.74

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

RUN #4

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
      ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                (All Rights Reserved-Unauthorized Use Prohibited)
  ************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
        Modified Bishop, Simplified Janbu, or GLE Method of Slices.
        (Includes Spencer & Morgenstern-Price Type Analysis)
        Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
        Nonlinear Undrained Shear Strength, Curved Phi Envelope,
        Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
        Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
  ************************************************************************
   Analysis Run Date:        2/24/2010
   Time of Run:              04:52PM
   Run By:                   KPC
   Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t3as1a.in
   Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t3as1a.OUT
   Unit System:              English
   Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t3as1a.PLT
   PROBLEM DESCRIPTION:  TT 53108, 3A-3A'; fill slope; static;
                         circular
   BOUNDARY COORDINATES
      8 Top   Boundaries
     16 Total Boundaries
   Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
     No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
      1          0.00      963.00      120.00      963.00         1
      2        120.00      963.00      338.00     1050.00         1
      3        338.00     1050.00      352.00     1050.00         1
      4        352.00     1050.00      358.00     1046.00         1
      5        358.00     1046.00      420.00     1046.00         1
      6        420.00     1046.00      425.00     1049.00         1
      7        425.00     1049.00      430.00     1052.00         5
      8        430.00     1052.00      600.00     1059.00         5
      9        425.00     1049.00      425.10     1025.00         1
     10        425.10     1025.00      600.00     1025.00         2
     11        425.10     1025.00      425.20      989.00         1
     12        425.20      989.00      600.00     1002.00         3
     13        425.20      989.00      425.30      980.00         3
     14        425.30      980.00      600.00      995.00         4
     15          0.00      950.00       70.00      952.00         4
     16         70.00      952.00      425.30      980.00         4
   User Specified Y-Origin =       900.00(ft)
   Default X-Plus Value = 0.00(ft)
   Default Y-Plus Value = 0.00(ft)
  ISOTROPIC SOIL PARAMETERS
    5 Type(s) of Soil
   Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
   Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
    No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
     1   125.0    130.0     300.0     30.0    0.12       0.0       0
     2   120.0    120.0     240.0     24.0    0.00       0.0       0
     3   115.0    120.0     250.0     31.0    0.00       0.0       0
     4   130.0    135.0     500.0     35.0    0.00       0.0       0
     5   120.0    120.0     240.0     24.0    0.00       0.0       0
   A Critical Failure Surface Searching Method, Using A Random
   Technique For Generating Circular Surfaces, Has Been Specified.
   2500 Trial Surfaces Have Been Generated.
     50 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
   Along The Ground Surface Between  X =  70.00(ft)
                                 and  X = 150.00(ft)
   Each Surface Terminates Between   X = 330.00(ft)
                                 and   X = 450.00(ft)
```

```
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =       0.00(ft)
20.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And  25.0 deg.
      Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
      Selected ki function = Bi-linear
      Selected Lambda Coefficient =  1.00
      The option of no water in the tension crack on
      the last slice has been selected.
      Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
      (if applicable) have been applied to the slice base(s)
      on which they intersect.
      Total Number of Trial Surfaces Evaluated =  2500
      WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
      Did Not Converge in 20 Iterations.
      Number of Trial Surfaces with Non-Converged FS =   17
      Percentage of Trial Surfaces With Non-Valid FS Solutions
      of the Total Evaluated =   0.7 %
      Statistical Data On All Valid FS Values:
         FS Max =  26.535   FS Min =   1.743   FS Ave =   3.254
         Standard Deviation =    0.957   Coefficient of Variation =   29.40 %
                 ((Modified Bishop FS for Critical Surface =  1.743))
      Failure Surface Specified By 14 Coordinate Points
        Point      X-Surf      Y-Surf
        No.         (ft)        (ft)
          1        118.98      963.00
          2        138.93      961.61
          3        158.93      961.64
          4        178.88      963.09
          5        198.67      965.94
          6        218.22      970.18
          7        237.41      975.80
          8        256.16      982.76
          9        274.37      991.03
         10        291.95     1000.57
         11        308.81     1011.33
         12        324.87     1023.25
         13        340.04     1036.28
         14        353.13     1049.24
      Circle Center At X =   148.55 ; Y =  1243.70 ; and Radius =   282.25
      ***  FOS =     1.743   Theta (ki=1.0) =    19.27  ***
                          Lambda =   0.350
```

Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.0	4.5	0.0	0.5	0.	0.	0.0	0.0	0.0
2	18.9	10663.7	0.0	1282.7	0.	0.	0.0	0.0	0.0
3	20.0	32294.9	0.0	3875.4	0.	0.	0.0	0.0	0.0
4	19.9	50250.8	0.0	6045.9	0.	0.	0.0	0.0	0.0
5	19.8	64176.7	0.0	7780.7	0.	0.	0.0	0.0	0.0
6	19.5	73870.5	0.0	9071.2	0.	0.	0.0	0.0	0.0
7	19.2	79265.6	0.0	9910.8	0.	0.	0.0	0.0	0.0
8	18.7	80430.2	0.0	10295.4	0.	0.	0.0	0.0	0.0
9	18.2	77566.9	0.0	10223.1	0.	0.	0.0	0.0	0.0
10	17.6	71005.9	0.0	9694.2	0.	0.	0.0	0.0	0.0
11	16.9	61197.1	0.0	8711.3	0.	0.	0.0	0.0	0.0
12	16.1	48701.1	0.0	7279.4	0.	0.	0.0	0.0	0.0
13	13.1	30346.4	0.0	4800.2	0.	0.	0.0	0.0	0.0
14	2.0	3724.3	0.0	589.1	0.	0.	0.0	0.0	0.0
15	12.0	11654.9	0.0	1969.1	0.	0.	0.0	0.0	0.0
16	1.1	132.9	0.0	22.4	0.	0.	0.0	0.0	0.0

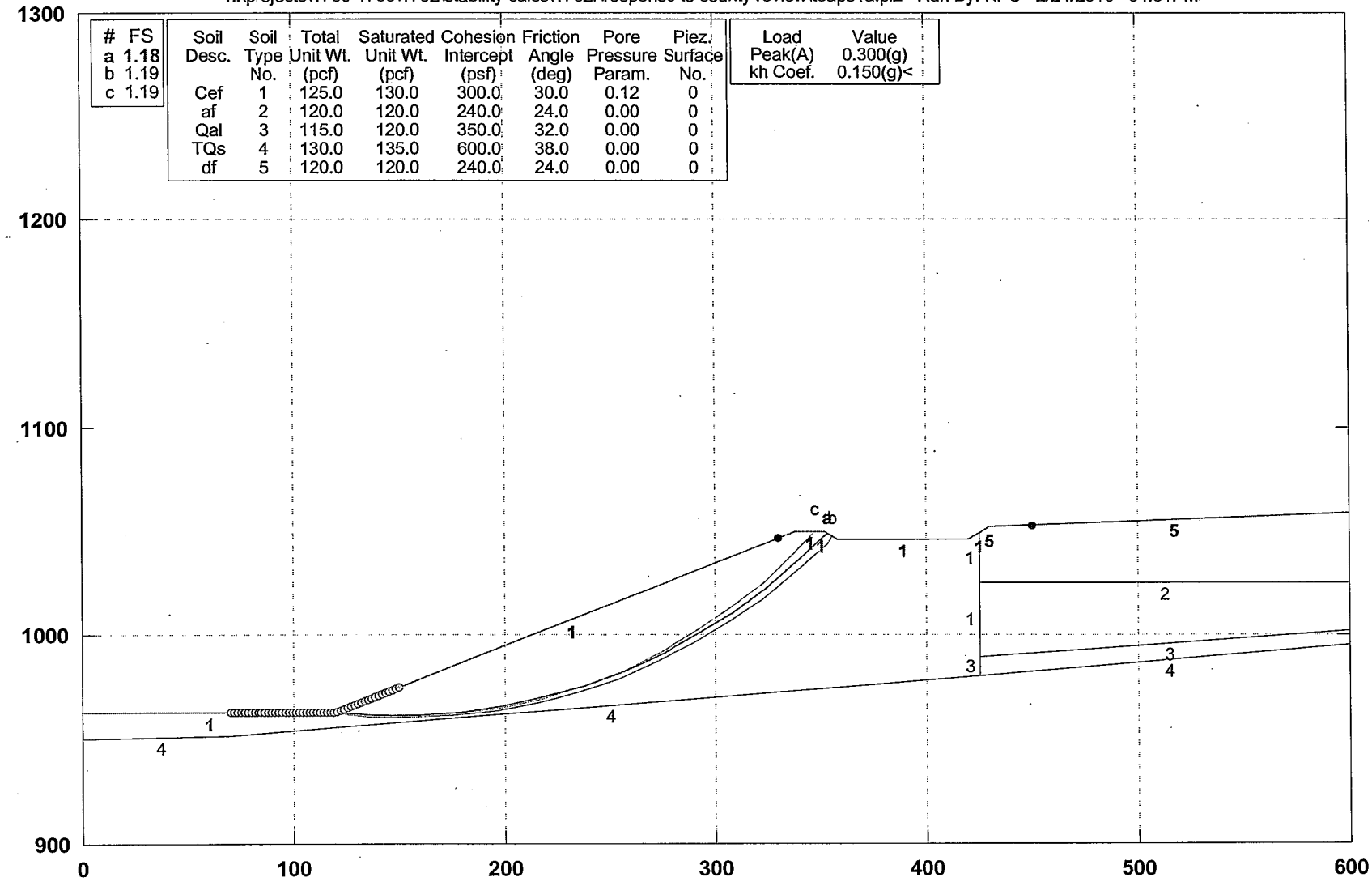
TT 53108, 3A-3A'; fill slope; pseudo; circular

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t3aps1a.pl2 Run By: KPC 2/24/2010 04:51PM

#	FS
a	1.18
b	1.19
c	1.19

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Piez. Surface No.
Cef	1	125.0	130.0	300.0	30.0	0.12	0
af	2	120.0	120.0	240.0	24.0	0.00	0
Qal	3	115.0	120.0	350.0	32.0	0.00	0
TQs	4	130.0	135.0	600.0	38.0	0.00	0
df	5	120.0	120.0	240.0	24.0	0.00	0

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.18

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

RUN # 5

```
                              *** GSTABL7 ***
                        ** GSTABL7 by Garry H. Gregory, P.E. **
          ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                      (All Rights Reserved-Unauthorized Use Prohibited)
     ********************************************************************************
                          SLOPE STABILITY ANALYSIS SYSTEM
               Modified Bishop, Simplified Janbu, or GLE Method of Slices.
               (Includes Spencer & Morgenstern-Price Type Analysis)
               Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
               Nonlinear Undrained Shear Strength, Curved Phi Envelope,
               Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
               Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
     ********************************************************************************
     Analysis Run Date:        2/24/2010
     Time of Run:              04:51PM
     Run By:                   KPC
     Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t3apsla.in
     Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t3apsla.OUT
     Unit System:              English
     Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t3apsla.PLT
     PROBLEM DESCRIPTION:  TT 53108, 3A-3A'; fill slope; pseudo;
                           circular
     BOUNDARY COORDINATES
         8 Top   Boundaries
        16 Total Boundaries
     Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
        No.        (ft)       (ft)       (ft)       (ft)    Below Bnd
         1          0.00     963.00     120.00     963.00        1
         2        120.00     963.00     338.00    1050.00        1
         3        338.00    1050.00     352.00    1050.00        1
         4        352.00    1050.00     358.00    1046.00        1
         5        358.00    1046.00     420.00    1046.00        1
         6        420.00    1046.00     425.00    1049.00        1
         7        425.00    1049.00     430.00    1052.00        5
         8        430.00    1052.00     600.00    1059.00        5
         9        425.00    1049.00     425.10    1025.00        1
        10        425.10    1025.00     600.00    1025.00        2
        11        425.10    1025.00     425.20     989.00        1
        12        425.20     989.00     600.00    1002.00        3
        13        425.20     989.00     425.30     980.00        3
        14        425.30     980.00     600.00     995.00        4
        15          0.00     950.00      70.00     952.00        4
        16         70.00     952.00     425.30     980.00        4
     User Specified Y-Origin =       900.00(ft)
     Default X-Plus Value = 0.00(ft)
     Default Y-Plus Value = 0.00(ft)
    ISOTROPIC SOIL PARAMETERS
      5 Type(s) of Soil
     Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
     Type Unit Wt. Unit Wt. Intercept  Angle  Pressure Constant Surface
      No.  (pcf)    (pcf)     (psf)    (deg)   Param.    (psf)     No.
       1   125.0    130.0     300.0     30.0    0.12       0.0       0
       2   120.0    120.0     240.0     24.0    0.00       0.0       0
       3   115.0    120.0     350.0     32.0    0.00       0.0       0
       4   130.0    135.0     600.0     38.0    0.00       0.0       0
       5   120.0    120.0     240.0     24.0    0.00       0.0       0
     Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
     Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
     Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
     Specified Seismic Pore-Pressure Factor =   0.000
     A Critical Failure Surface Searching Method, Using A Random
     Technique For Generating Circular Surfaces, Has Been Specified.
     2500 Trial Surfaces Have Been Generated.
       50 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
```

```
Along The Ground Surface Between  X =  70.00(ft)
                            and  X = 150.00(ft)
Each Surface Terminates Between   X = 330.00(ft)
                            and   X = 450.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is  Y =      0.00(ft)
20.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -45.0
And  25.0 deg.

      Following Are Displayed The Ten Most Critical Of The Trial
      Failure Surfaces Evaluated. They Are
      Ordered - Most Critical First.
      * * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
      Selected ki function = Bi-linear
      Selected Lambda Coefficient =  1.00
      The option of no water in the tension crack on
      the last slice has been selected.
      Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
      (if applicable) have been applied to the slice base(s)
      on which they intersect.
      Total Number of Trial Surfaces Evaluated =  2500
      WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
      Did Not Converge in 20 Iterations.
      Number of Trial Surfaces with Non-Converged FS =   17
      Percentage of Trial Surfaces With Non-Valid FS Solutions
      of the Total Evaluated =  0.7 %
      Statistical Data On All Valid FS Values:
         FS Max =  10.720   FS Min =   1.184   FS Ave =   2.188
         Standard Deviation =    0.534   Coefficient of Variation =   24.41 %
                 ((Modified Bishop FS for Critical Surface =  1.203))
      Failure Surface Specified By 14 Coordinate Points
        Point      X-Surf      Y-Surf
        No.         (ft)        (ft)
          1        118.98      963.00
          2        138.93      961.61
          3        158.93      961.64
          4        178.88      963.09
          5        198.67      965.94
          6        218.22      970.18
          7        237.41      975.80
          8        256.16      982.76
          9        274.37      991.03
         10        291.95     1000.57
         11        308.81     1011.33
         12        324.87     1023.25
         13        340.04     1036.28
         14        353.13     1049.24
      Circle Center At X =   148.55 ; Y =  1243.70 ; and Radius =   282.25
      ***  FOS =    1.184   Theta (ki=1.0) =    18.60  ***
                         Lambda =   0.337
```

Individual data on the 16 slices

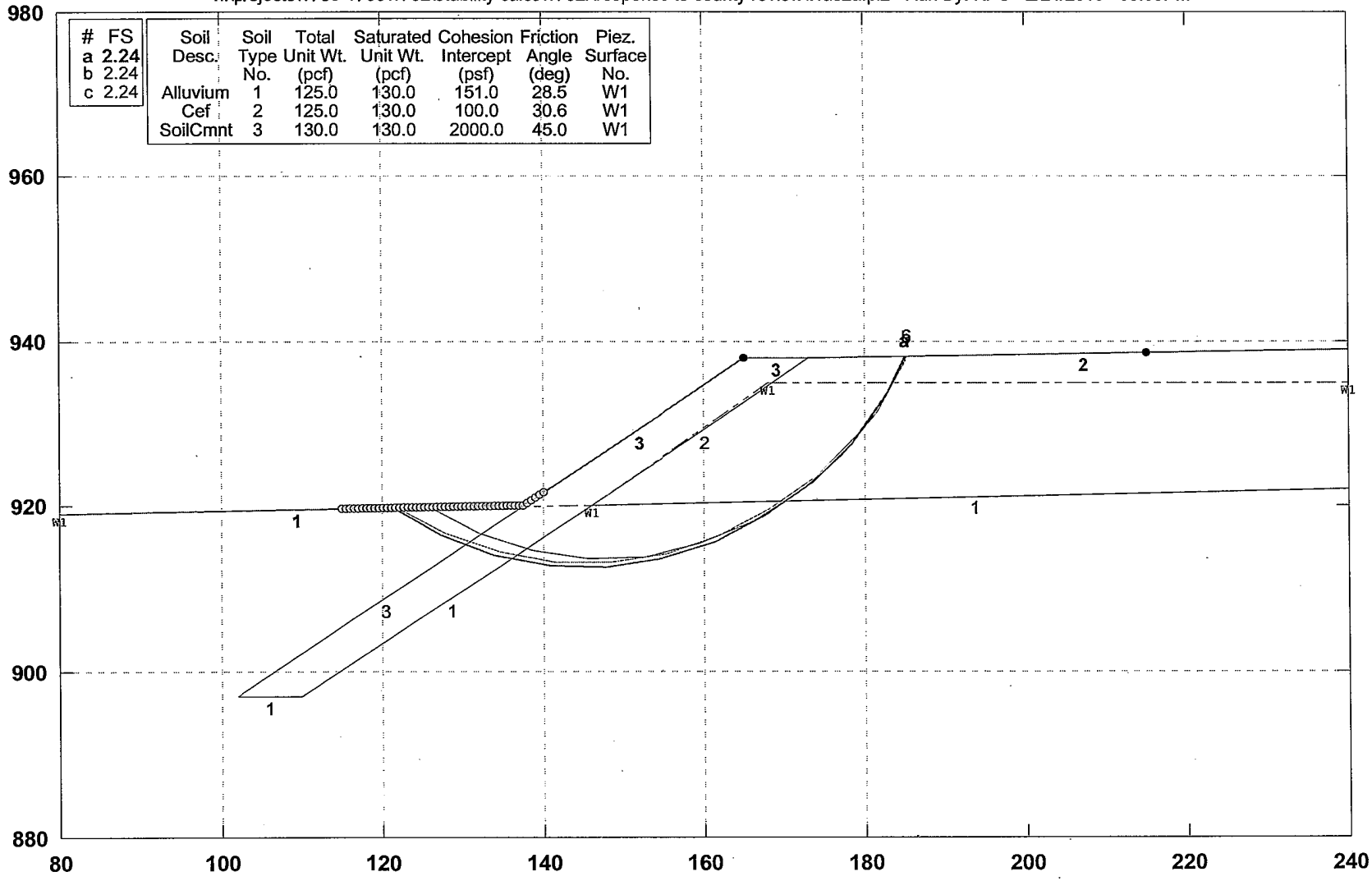
Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.0	4.5	0.0	0.5	0.	0.	0.7	0.0	0.0
2	19.9	10663.7	0.0	1282.7	0.	0.	1599.6	0.0	0.0
3	20.0	32294.9	0.0	3875.4	0.	0.	4844.2	0.0	0.0
4	19.9	50250.8	0.0	6045.9	0.	0.	7537.6	0.0	0.0
5	19.8	64176.7	0.0	7780.7	0.	0.	9626.5	0.0	0.0
6	19.5	73870.5	0.0	9071.2	0.	0.	11080.6	0.0	0.0
7	19.2	79265.6	0.0	9910.8	0.	0.	11889.8	0.0	0.0
8	18.7	80430.2	0.0	10295.4	0.	0.	12064.5	0.0	0.0
9	18.2	77566.9	0.0	10223.1	0.	0.	11635.0	0.0	0.0
10	17.6	71005.9	0.0	9694.2	0.	0.	10650.9	0.0	0.0
11	16.9	61197.1	0.0	8711.3	0.	0.	9179.6	0.0	0.0
12	16.1	48701.1	0.0	7279.4	0.	0.	7305.2	0.0	0.0
13	13.1	30346.4	0.0	4800.2	0.	0.	4552.0	0.0	0.0
14	2.0	3724.3	0.0	589.1	0.	0.	558.7	0.0	0.0
15	12.0	11654.9	0.0	1968.1	0.	0.	1748.2	0.0	0.0
16	1.1	132.9	0.0	22.4	0.	0.	19.9	0.0	0.0

Lndmrk Village Bank Pro.; 4A-4A'; rapid drawdown; static; circular

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t4as2a.pl2 Run By: KPC 2/24/2010 05:09PM

#	FS
a	2.24
b	2.24
c	2.24

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Alluvium	1	125.0	130.0	151.0	28.5	W1
Cef	2	125.0	130.0	100.0	30.6	W1
SoilCmnt	3	130.0	130.0	2000.0	45.0	W1



GSTABL7 v.2 FSmin=2.24

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run #6

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
      ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                (All Rights Reserved-Unauthorized Use Prohibited)
   ************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
         Modified Bishop, Simplified Janbu, or GLE Method of Slices.
         (Includes Spencer & Morgenstern-Price Type Analysis)
         Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
         Nonlinear Undrained Shear Strength, Curved Phi Envelope,
         Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
         Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
   ************************************************************************
    Analysis Run Date:        2/24/2010
    Time of Run:              05:09PM
    Run By:                   KPC
    Input Data Filename:      n:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t4as2a.in
    Output Filename:          n:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t4as2a.OUT
    Unit System:              English
    Plotted Output Filename:  n:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t4as2a.PLT
    PROBLEM DESCRIPTION:  Lndmrk Village Bank Pro.; 4A-4A';
                          rapid drawdown; static; circular
    BOUNDARY COORDINATES
        4 Top   Boundaries
        9 Total Boundaries
    Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
       No.        (ft)        (ft)        (ft)        (ft)     Below Bnd
        1        80.00      919.00      137.50      920.00         1
        2       137.50      920.00      165.00      938.00         3
        3       165.00      938.00      173.00      938.00         3
        4       173.00      938.00      240.00      939.00         2
        5       102.00      897.00      137.50      920.00         3
        6       102.00      897.00      110.00      897.00         1
        7       110.00      897.00      146.00      920.00         1
        8       146.00      920.00      173.00      938.00         2
        9       146.00      920.00      240.00      922.00         1
    User Specified Y-Origin =       880.00(ft)
    Default X-Plus Value = 0.00(ft)
    Default Y-Plus Value = 0.00(ft)
   ISOTROPIC SOIL PARAMETERS
     3 Type(s) of Soil
    Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
    Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
     No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
      1   125.0    130.0     151.0     28.5    0.00       0.0      1
      2   125.0    130.0     100.0     30.6    0.00       0.0      1
      3   130.0    130.0    2000.0     45.0    0.00       0.0      1
    1 PIEZOMETRIC SURFACE(S) SPECIFIED
    Unit Weight of Water =  62.40 (pcf)
    Piezometric Surface No.  1 Specified by  4 Coordinate Points
    Pore Pressure Inclination Factor =  0.50
      Point      X-Water     Y-Water
       No.         (ft)        (ft)
        1         80.00      918.90
        2        146.00      920.00
        3        168.00      935.00
        4        240.00      935.00
    A Critical Failure Surface Searching Method, Using A Random
    Technique For Generating Circular Surfaces, Has Been Specified.
    2500 Trial Surfaces Have Been Generated.
      50 Surface(s) Initiate(s) From Each Of    50 Points Equally Spaced
    Along The Ground Surface Between  X = 115.00(ft)
                                  and  X = 140.00(ft)
    Each Surface Terminates Between   X = 165.00(ft)
```

and X = 215.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
7.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -35.0
And 25.0 deg.
Factor Of Safety Calculation By GLE (Spencer's) Method of Slices
Has Gone Through 20 Iterations

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
15.00	3.396	4.189	0.268
22.50	3.152	4.634	0.414
3.50	3.591	3.706	0.061
0.00	3.622	3.589	0.000
0.00	3.622	3.589	0.000
12.49	3.453	4.068	0.222
0.86	3.615	3.617	0.015
0.00	3.622	3.589	0.000
0.82	3.616	3.615	0.014
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015
0.83	3.616	3.616	0.015

##################### SOME LINES SKIPPED #####################
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 117
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 4.7 %
Statistical Data On All Valid FS Values:
FS Max = 26.514 FS Min = 2.237 FS Ave = 2.861
Standard Deviation = 1.285 Coefficient of Variation = 44.92 %
((Modified Bishop FS for Critical Surface = 2.178))
Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	121.12	919.72
2	127.26	916.36
3	133.86	914.02
4	140.75	912.76
5	147.75	912.61
6	154.68	913.57
7	161.37	915.63
8	167.65	918.72
9	173.36	922.77
10	178.35	927.67

11 182.50 933.31
12 185.00 938.18
Circle Center At X = 145.16 ; Y = 956.29 ; and Radius = 43.77
*** FOS = 2.237 Theta (ki=1.0) = 12.53 ***
Lambda = 0.222
Individual data on the 23 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.2	1.9	0.0	0.0	0.	0.	0.0	0.0	0.0
2	5.9	1377.4	0.0	700.0	0.	0.	0.0	0.0	0.0
3	3.0	1555.8	0.0	766.5	0.	0.	0.0	0.0	0.0
4	3.6	2463.6	0.0	1222.8	0.	0.	0.0	0.0	0.0
5	2.2	1718.8	0.0	820.2	0.	0.	0.0	0.0	0.0
6	1.3	1056.9	0.0	504.9	0.	0.	0.0	0.0	0.0
7	0.2	193.2	0.0	92.3	0.	0.	0.0	0.0	0.0
8	3.2	3380.9	0.0	1407.0	0.	0.	0.0	0.0	0.0
9	5.3	7608.5	0.0	2377.9	0.	0.	0.0	0.0	0.0
10	1.7	3055.4	0.0	730.6	0.	0.	0.0	0.0	0.0
11	6.9	14263.3	0.0	3845.5	0.	0.	0.0	0.0	0.0
12	6.7	16319.8	0.0	4997.9	0.	0.	0.0	0.0	0.0
13	3.6	9534.1	0.0	3227.2	0.	0.	0.0	0.0	0.0
14	2.7	6842.3	0.0	2459.4	0.	0.	0.0	0.0	0.0
15	0.3	864.1	0.0	359.8	0.	0.	0.0	0.0	0.0
16	0.5	1225.7	0.0	606.6	0.	0.	0.0	0.0	0.0
17	1.7	3954.1	0.0	1943.4	0.	0.	0.0	0.0	0.0
18	2.8	6006.2	0.0	2905.7	0.	0.	0.0	0.0	0.0
19	0.4	714.6	0.0	340.9	0.	0.	0.0	0.0	0.0
20	5.0	8245.5	0.0	4271.3	0.	0.	0.0	0.0	0.0
21	4.1	4044.3	0.0	1968.4	0.	0.	0.0	0.0	0.0
22	0.9	436.6	0.0	99.9	0.	0.	0.0	0.0	0.0
23	1.6	322.3	0.0	0.0	0.	0.	0.0	0.0	0.0

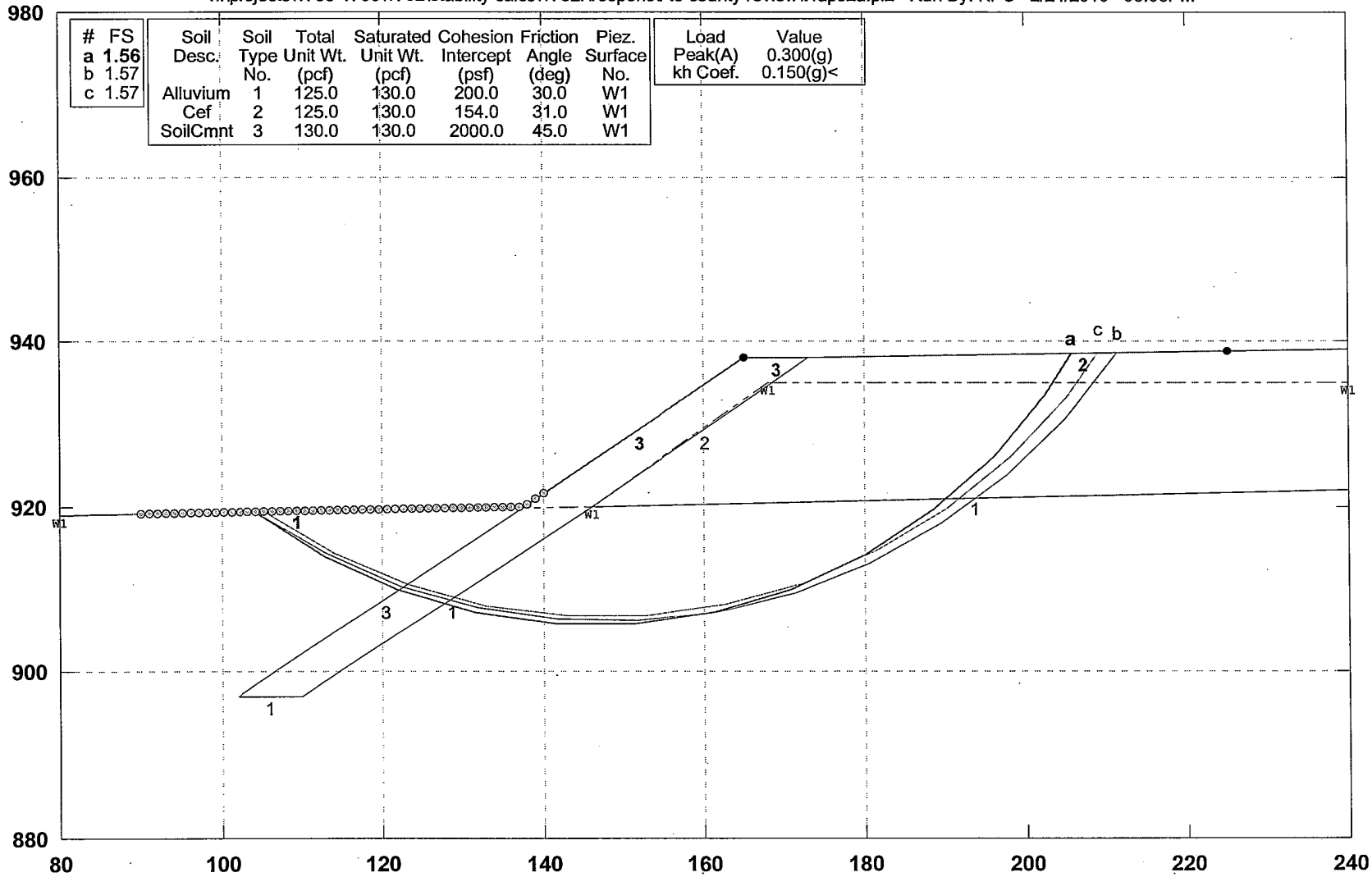
Lndmrk Village Bank Pro.; 4A-4A'; rapid drawdown; pseudo; circular

n:\projects\1700-1799\1702\stability calcs\1702r\response to county review\t4aps2a.pl2 Run By: KPC 2/24/2010 05:06PM

#	FS
a	1.56
b	1.57
c	1.57

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Alluvium	1	125.0	130.0	200.0	30.0	W1
Cef	2	125.0	130.0	154.0	31.0	W1
SoilCmnt	3	130.0	130.0	2000.0	45.0	W1

Load	Value
Peak(A)	0.300(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.56

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

Run #7

```
                    *** GSTABL7 ***
                ** GSTABL7 by Garry H. Gregory, P.E. **
    ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                (All Rights Reserved-Unauthorized Use Prohibited)
 ************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
        Modified Bishop, Simplified Janbu, or GLE Method of Slices.
        (Includes Spencer & Morgenstern-Price Type Analysis)
        Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
        Nonlinear Undrained Shear Strength, Curved Phi Envelope,
        Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
        Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
 ************************************************************************
 Analysis Run Date:        2/24/2010
 Time of Run:              05:06PM
 Run By:                   KPC
 Input Data Filename:      N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t4aps2a.in
 Output Filename:          N:\Projects\1700-1799\1702\Stability Calcs\1702R\Response to
County Review\t4aps2a.OUT
 Unit System:              English
 Plotted Output Filename:  N:\Projects\1700-1799\1702\Stabi Calcs\1702R\Response to County
Review\t4aps2a.PLT
 PROBLEM DESCRIPTION:  Lndmrk Village Bank Pro.; 4A-4A';
                       rapid drawdown; pseudo; circular
 BOUNDARY COORDINATES
    4 Top   Boundaries
    9 Total Boundaries
 Boundary     X-Left      Y-Left     X-Right     Y-Right     Soil Type
   No.         (ft)        (ft)        (ft)        (ft)     Below Bnd
    1         80.00      919.00      137.50      920.00         1
    2        137.50      920.00      165.00      938.00         3
    3        165.00      938.00      173.00      938.00         3
    4        173.00      938.00      240.00      939.00         2
    5        102.00      897.00      137.50      920.00         3
    6        102.00      897.00      110.00      897.00         1
    7        110.00      897.00      146.00      920.00         1
    8        146.00      920.00      173.00      938.00         2
    9        146.00      920.00      240.00      922.00         1
 User Specified Y-Origin =       880.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
 ISOTROPIC SOIL PARAMETERS
  3 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)   Param.    (psf)     No.
   1   125.0    130.0     200.0     30.0    0.00       0.0      1
   2   125.0    130.0     154.0     31.0    0.00       0.0      1
   3   130.0    130.0    2000.0     45.0    0.00       0.0      1
 1 PIEZOMETRIC SURFACE(S) SPECIFIED
 Unit Weight of Water =  62.40 (pcf)
 Piezometric Surface No.  1 Specified by  4 Coordinate Points
 Pore Pressure Inclination Factor =  0.50
   Point      X-Water     Y-Water
    No.         (ft)        (ft)
     1         80.00      918.90
     2        146.00      920.00
     3        168.00      935.00
     4        240.00      935.00
 Specified Peak Ground Acceleration Coefficient (A) =   0.300(g)
 Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
 Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
 Specified Seismic Pore-Pressure Factor =   0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 2500 Trial Surfaces Have Been Generated.
```

50 Surface(s) Initiate(s) From Each Of 50 Points Equally Spaced
Along The Ground Surface Between X = 90.00(ft)
and X = 140.00(ft)
Each Surface Terminates Between X = 165.00(ft)
and X = 225.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
10.00(ft) Line Segments Define Each Trial Failure Surface.
Restrictions Have Been Imposed Upon The Angle Of Initiation.
The Angle Has Been Restricted Between The Angles Of -35.0
And 25.0 deg.

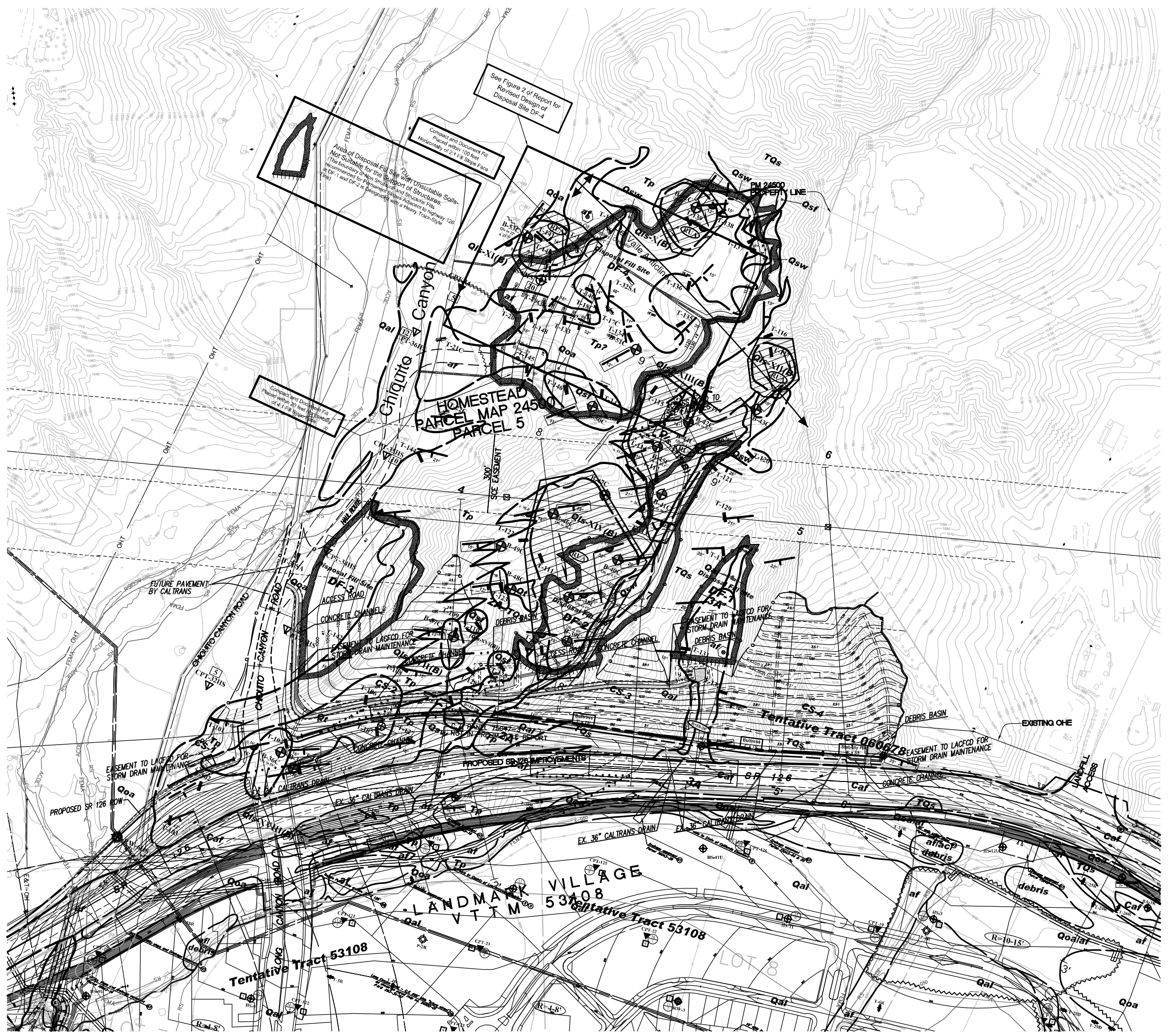
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By GLE (Spencer's) Method (0-2) * *
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
The option of no water in the tension crack on
the last slice has been selected.
Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.
Total Number of Trial Surfaces Evaluated = 2500
WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.
Number of Trial Surfaces with Non-Converged FS = 235
Number of Trial Surfaces with Misleading FS = 16
Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Evaluated = 10.0 %
Statistical Data On All Valid FS Values:
FS Max = 17.678 FS Min = 1.564 FS Ave = 1.978
Standard Deviation = 0.960 Coefficient of Variation = 48.52 %
((Modified Bishop FS for Critical Surface = 1.633))
Failure Surface Specified By 13 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	104.29	919.42
2	112.72	914.05
3	121.83	909.93
4	131.44	907.14
5	141.34	905.74
6	151.34	905.75
7	161.23	907.18
8	170.83	910.00
9	179.93	914.15
10	188.35	919.54
11	195.92	926.07
12	202.49	933.61
13	205.65	938.49

Circle Center At X = 146.24 ; Y = 976.01 ; and Radius = 70.44
*** FOS = 1.564 Theta (ki=1.0) = 7.35 ***
Lambda = 0.129
Individual data on the 24 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.2	1.3	0.0	0.0	0.	0.	0.2	0.0	0.0
2	8.3	3018.7	0.0	1646.7	0.	0.	452.8	0.0	0.0
3	9.1	9063.8	0.0	4698.3	0.	0.	1359.6	0.0	0.0
4	0.1	110.0	0.0	54.3	0.	0.	16.5	0.0	0.0
5	5.7	7914.0	0.0	3908.2	0.	0.	1187.1	0.0	0.0
6	3.8	6046.2	0.0	2989.3	0.	0.	906.9	0.0	0.0
7	5.8	10032.5	0.0	4813.7	0.	0.	1504.9	0.0	0.0
8	0.2	399.7	0.0	191.8	0.	0.	59.9	0.0	0.0
9	3.8	7603.5	0.0	3355.6	0.	0.	1140.5	0.0	0.0
10	4.7	11092.7	0.0	4137.7	0.	0.	1663.9	0.0	0.0

11	5.3	14891.0	0.0	4502.3	0.	0.	2233.6	0.0	0.0
12	9.9	33078.8	0.0	10786.2	0.	0.	4961.8	0.0	0.0
13	3.8	14149.0	0.0	4932.7	0.	0.	2122.4	0.0	0.0
14	3.0	11360.7	0.0	4144.5	0.	0.	1704.1	0.0	0.0
15	0.5	1869.3	0.0	837.6	0.	0.	280.4	0.0	0.0
16	2.3	8570.6	0.0	3837.8	0.	0.	1285.6	0.0	0.0
17	2.2	7740.1	0.0	3649.3	0.	0.	1161.0	0.0	0.0
18	6.9	22846.3	0.0	10657.5	0.	0.	3426.9	0.0	0.0
19	8.4	23208.0	0.0	11329.8	0.	0.	3481.2	0.0	0.0
20	1.6	3758.7	0.0	1967.4	0.	0.	563.8	0.0	0.0
21	6.0	11353.5	0.0	5641.4	0.	0.	1703.0	0.0	0.0
22	6.6	7191.1	0.0	3218.8	0.	0.	1078.7	0.0	0.0
23	0.9	468.9	0.0	71.7	0.	0.	70.3	0.0	0.0
24	2.3	487.6	0.0	0.0	0.	0.	73.1	0.0	0.0



LEGEND:

- EXISTING LOT LINE
- EXISTING EASEMENT
- PROPOSED CONTOUR (1100)
- DAYLIGHT LINE
- CUT & FILL
- FLOW LINE
- BENCH
- SWALE
- HAUL ROUTE ACCESS ROAD
- RIVER CORRIDOR–2003 (FORMERLY SEA 23) (RC03)
- RIVER CORRIDOR (FORMERLY SEA 23) (RC)
- FEMA
- Q–CAP (Q50)
- DEBRIS TRIBUTARY LIMIT
- SLOPE/ DEBRIS BASIN ACCESS
- DEBRIS BASIN
- OIL WELL GPS POSITIONED
- OIL WELL DEPT. OF OIL AND GAS CALCULATED POSITION
- OIL WELL DIGITIZED POSITION
- OIL WELL DEPT. OF OIL AND GAS METAL DETECTOR POSITION
- EXISTING OAK TREE
- RIP–RAP
- SPINE FLOWER
- SPINE FLOWER PROTECTION BOUNDARY

FEET: 200 100 0 200 400
INCHES: 1 1/2 0 1 2

OFFSITE GRADING
CHIQUITO CANYON

CHIQUITO CANYON EARTHWORK VOLUMES	
RAW CUT	1,190,000 Cubic Yards
RAW FILL	1,071,000 Cubic Yards
NET CUT	1,190,000 Cubic Yards
NET FILL	1,190,000 Cubic Yards
EXPORT	0 Cubic Yards

FEET: 200 100 0 200 400
INCHES: 1 1/2 0 1 2

OFFSITE GRADING
ADOBE CANYON

ADOBE CANYON EARTHWORK VOLUMES	
RAW CUT	6,468,500 Cubic Yards
RAW FILL	133,000 Cubic Yards
NET CUT	6,468,500 Cubic Yards
NET FILL	146,300 Cubic Yards
EXPORT	6,322,200 Cubic Yards

REV. DATE: DECEMBER 21, 2009
DATE: MAY 2, 2007

ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological And Geotechnical Consultants

Geologic/Geotechnical Map

Date: 12/21/09 | Job No.: 09-1702R-4
Scale: 1" = 200' | CAD File: Sheet 11 Addendum Geologic Map
Sheet 11 | Geology by: BJS | Drawn by: GSD/JCH | Revised: 2/25/10

SEE 9/30/05 REPORT FOR VTT 060678 AND SHEET 3 FOR GEOLOGIC LEGEND

LEGAL DESCRIPTION:

PARCELS 8, 9, 10, AND PORTION OF PARCELS 7, 20, 21 OF PARCEL MAP NO. 24500-01 PMB 293-34/67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28480 Avenue Stanford, Suite 200
Santa Clarita, CA 91355
(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C 67285 DATE

DESIGNED: NSK/YL
DRAFTED: TLC/JS
CHECKED: EW

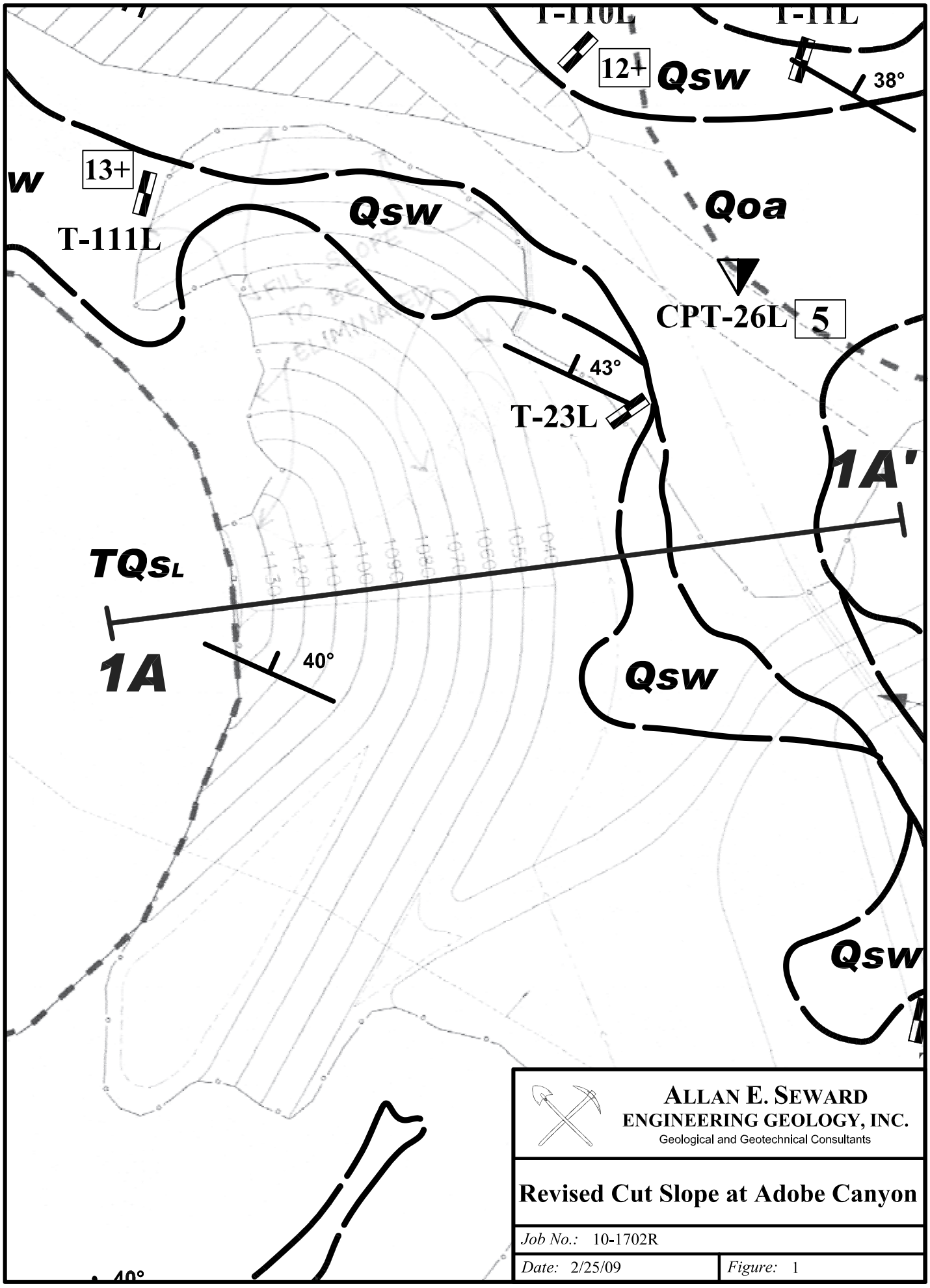
OWNER/DEVELOPER:
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE: MR. FRED MACMURDO

MAJOR LAND DIVISION
VESTING TENTATIVE TRACT MAP
NO. 53108

CHIQUITO CANYON & ADOBE CANYON OFFSITE GRADING
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NLF010500
SHEET 11 OF 13

RV001-11.dwg

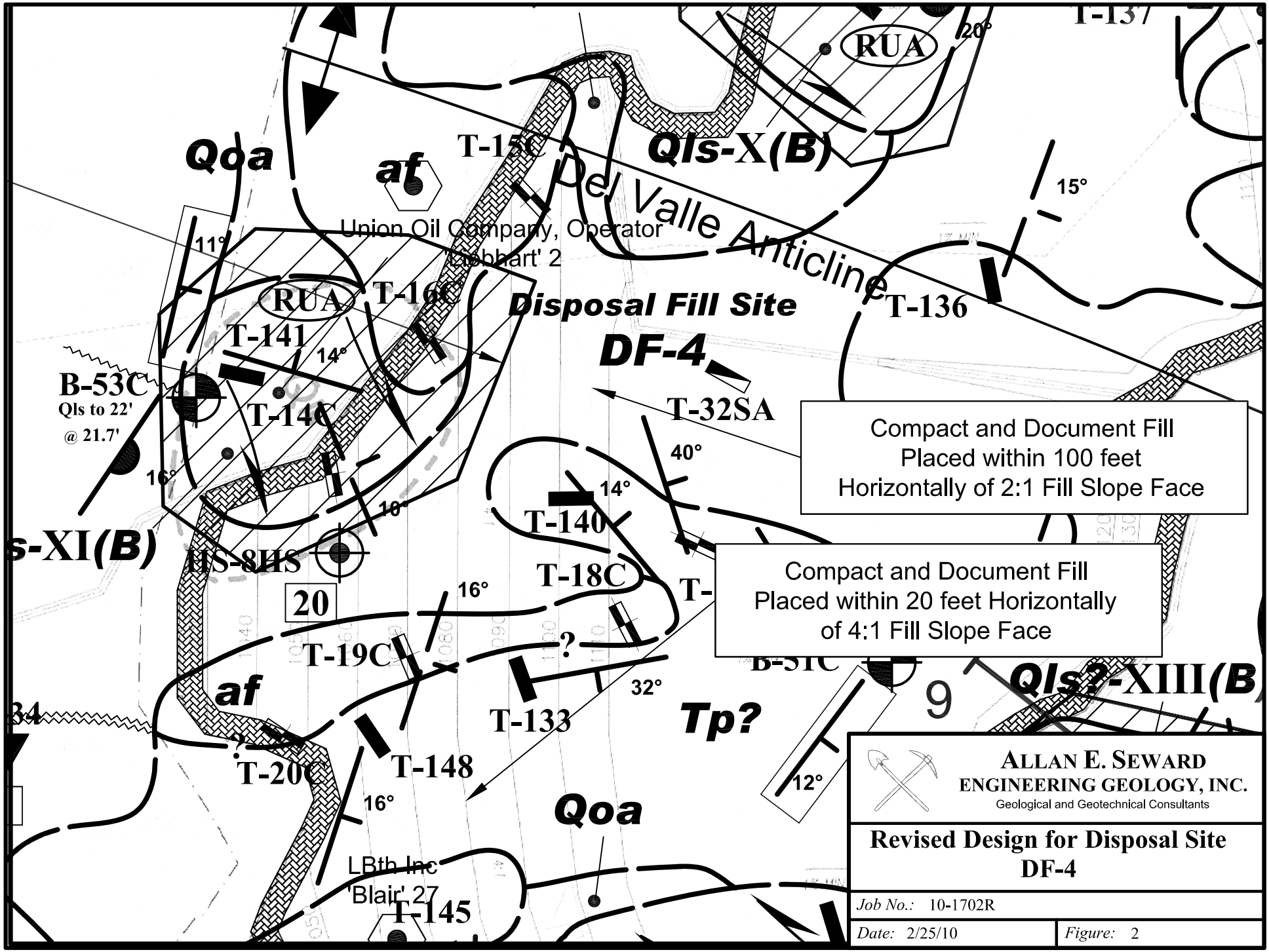


ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants

Revised Cut Slope at Adobe Canyon

Job No.: 10-1702R

Date: 2/25/09 | *Figure:* 1


Qoa
af
T-15C
Qls-X(B)
Del Valle Anticline
RUA
20°
T-137
15°
11°
Union Oil Company, Operator
'Ebbhart' 2
RUA
T-16C
Disposal Fill Site
T-136
T-141
14°
DF-4
B-53C
Qls to 22'
@ 21.7'
T-14C
T-32SA
16°
40°
14°
10°
T-140
s-XI(B)
HS-8HS
T-18C
20
16°
T-19C
?
32°
af
T-133
Tp?
Qls?-XIII(B)
9
T-20C
T-148
16°
12°
Qoa
LBth Inc
'Blair' 27
T-145
Compact and Document Fill
Placed within 100 feet
Horizontally of 2:1 Fill Slope Face
Compact and Document Fill
Placed within 20 feet Horizontally
of 4:1 Fill Slope Face
ALLAN E. SEWARD
ENGINEERING GEOLOGY, INC.
Geological and Geotechnical Consultants
Revised Design for Disposal Site
DF-4
Job No.: 10-1702R
Date: 2/25/10
Figure: 2

Appendix B

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

February 23, 2010

Newhall Land and Farming Company
25124 Springfield Court, Suite 300
Valencia, California 91355

Job No. 99-802-022

Attention: Mr. Fred MacMurdo

Subject: Response to County Comment
Proposed Offsite Zone 1A Water Tank Site
Vesting Tentative Tract Map No. 53108
Castaic, Los Angeles County, California

Reference: Geotechnical Grading Plan Review
Proposed Offsite Zone 1A Water Tank Site
Vesting Tentative Tract Map No. 53108
Los Angeles County, California
Dated March 28, 2008, Job No. 99-802-022

Ladies/Gentlemen:

This submittal provides information requested by the County of Los Angeles Department of Public Works (LACDPW). The LACDPW comments were prepared following their review of the Allan E. Seward Engineering Geology, Inc. (AES) report dated December 21, 2009. The Soils Engineering Review Sheet has a date of February 5, 2009, but it is assumed to be actually issued on February 5, 2010 and a hand change has been made on the review sheet attached to this report. Our referenced report, dated March 28, 2008, was presented within Appendix D of the AES report. Comment No. 6 was the only comment related to the review of our referenced report. This response only includes responding to Comment No. 6 of the February 5, 2009 review comments. We understand that the remainder of the review comments will be addressed by AES.

R. T. FRANKIAN & ASSOCIATES
1329 SCOTT ROAD BURBANK CALIFORNIA 91504
TEL. (818) 531-1501 FAX (818) 531-1511 WWW.RTFRANKIAN.COM

SOILS ENGINEERING REVIEW SHEET

1. **Provide static and seismic slope stability analyses for the proposed cut slopes surrounding the Zone 1A Water Tank at PM 18108. The referenced R. T. Frankian Report dated 3/28/08 does not include slope stability analyses. Also, provide a geotechnical cross section, for each section analyzed, showing the critical failure plane used in the analyses. Indicate the various shear strength parameters used in the analyses, in the appropriate segments of each failure plane. Show locations of the cross sections used in slope stability analyses on the geotechnical map. Recommend mitigation if factors of safety are below County minimum standards.**

The slope stability calculations presented in this response all utilized Saugus Cross Bedding shear strength parameters for cohesion of 640 psf and an angle of internal friction of 32 degrees. The referenced report recommended the following shear strength parameters for the subject site.

SUMMARY OF SHEAR STRENGTH PARAMETERS

MATERIAL	Static Cohesion (psf)	Static ϕ (degrees)
Landslide Failure Plane Material	200	16
Clayey Bedding Plane Material	250	20
Clayey Bedding Plane Material (Seismic)	375	19
Saugus Cross Bedding	640	32
Alluvium	100	40
Compacted Fill	300	32

The Saugus Formation can range from massive to thinly bedded sedimentary rock units of sandstone, conglomerate, siltstone, and claystone/mudstone. Bedding planes within the Saugus Formation are poorly to moderately-well developed, and can constitute planes of weakness, particularly where sandstone/conglomerate beds are in contact with claystone. Where bedding is adversely oriented, or "daylighted," with

respect to natural or cut slopes, there is potential for bedding plane, or "block-glide," failure.

The subsurface geologic conditions are depicted on previously presented Geologic Sections A-A' and B-B,' which are presented as Figures 2.1 and 2.2, respectively, for this submittal. The locations of the Geologic Sections are shown on the Geotechnical Map, Figure 1.

Geologic Section A-A' illustrates geologic conditions parallel to the axis of the easterly-plunging syncline, for the segment of cut slope CS-1 facing to the east. Bedding in this area strikes northeast and dips 40 degrees to 45 degrees to the southeast. The apparent dip, with respect to the east-facing cut slope segment, is 32 degrees towards the east. The proposed east-facing segment of the cut slope is considered to be grossly stable relative to bedding plane failures, as the angle of apparent dip (32 degrees) is steeper than the proposed 2:1 (or approximately 26 degrees) cut slope. Slope stability calculations for a circular cross-bedding failure are presented in the Appendix and exceed the county minimum factor of safety requirements. The critical failure surface, material strength properties, and the calculated factor of safety are shown on Geotechnical Section A-A' presented in Figure 3.1.

Geologic Section B-B' depicts the geologic conditions for cut slope CS-1 perpendicular to the synclinal axis, and across the south-facing portion of the cut-slope. Bedding with respect to the south-facing segment of cut slope CS-1 exhibits an apparent dip of 37 degrees to the south. This bedding orientation dips steeper than the 2:1 (26 degrees) cut slope gradient, and the south-facing segment of cut slope CS-1 is considered grossly stable relative to bedding plane failures. Slope stability calculations for a circular cross-bedding failure are presented in the Appendix, and exceed the county minimum factor of safety requirements. The critical failure surface, material strength properties, and the calculated factor of safety are shown on Geotechnical Section B-B' presented in Figure 3.1.

The following are attached and complete this response:

- Soils Engineering Review Sheet dated February 5, 2009
- Geotechnical Map, Figure 1
- Geologic Cross Section A-A', Figure 2.1
- Geologic Cross Section B-B', Figure 2.2
- Geotechnical Section A-A', Figure 3.1
- Geotechncial Section B-B', Figure 3.2
- Appendix - Slope Stability Calculations (4 pages)

Yours very truly,

R. T. FRANKIAN & ASSOCIATES

by: Alan W. Rasplicka
Principal Geotechnical Engineer

and: Timothy P. Latiolait
Principal Engineering Geologist

Distribution: (5) Newhall Land & Farming Company
Mr. Fred MacMurdo
("wet signed," stamped, and unbound copies and a CD with a PDF file for regulatory submittal)

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL AND MATERIALS ENGINEERING DIVISION

SOILS ENGINEERING REVIEW SHEET

Address: 900 S. Fremont Ave., Alhambra, CA 91803
Telephone: (626) 458-4925
Fax: (626) 458-4913

District Office 8.2
Job Number GMPH
Sheet 1 of 1

DISTRIBUTION:
___ Drainage
___ Grading
___ Geo/Soils Central File
___ District Engineer
___ Geologist
___ Soils Engineer
___ Engineer/Architect

Tentative Tract Map	53108
Location	Newhall Ranch
Developer/Owner	Newhall Land and Farming Company / Newhall Ranch Co.
Engineer/Architect	Psomas
Soils Engineer	Allan E. Seward Engineering Geology, Inc.
Geologist	Same as above

Review of:
Revised Tentative Tract Map and Exhibit Dated 5/2/07
Soils Engineering and Geologic Reprot Dated **12/21/09**, 10/29/03, 2/10/01, 9/27/00
Previous Review Sheet Dated 11/13/08

ACTION:

Tentative Map feasibility is not recommended for approval.

REMARKS:

1. The geotechnical map within the submitted report does not match the latest approved Tentative Tract Map dated by Regional Planning 5/2/07. Provide revised Tentative Tract Map dated by Regional Planning and geotechnical map that conform.

2. Show the final graded contours of all landslides to be removed and recompacted on the geotechnical maps and Tentative Map. For example, Landslide Qls XVI(B) is recommended to removed and recompacted, however the final graded contours are not shown on the geotechnical map or the Tentative Map. Revise geotechnical maps and Tentative Map as necessary.

3. On the geotechnical map, delineate approximate limits of proposed disposal fills. Also delineate all soils unsuitable for the support of structures (proposed disposal fill, landslide debris) and prominently note/label with the note: "Unsuitable Soils - Not suitable for the Support of Structures".

4. Discuss and address whether the proposed fill slopes associated with Disposal Fill areas will be subject to erosion and/or slope instability. Provide updated surficial, static, and seismic slope stability analyses considering the density, moisture content, and shear strengths of the disposal fill materials as well as the proposed design shown on the latest geotechnical map and Tentative Map. Provide additional recommendations as necessary.

5. Proposed Disposal Fill areas DF-1 and DF-2 appear to show debris basins located in disposal fill. Clarify the soil material that will be used to construct the proposed debris basins and provide recommendations as necessary.

6. Provide static and seismic slope stability analyses for the proposed cut slopes surrounding the Zone 1A Water Tank at PM 18108. The referenced R.T. Frankian Report dated 3/28/08 does not include slope stability analyses. Also, provide a geotechnical cross section, for each section analyzed, showing the critical failure plane used in the analyses. Indicate the various shear strength parameters used in the analyses, in the appropriate segments of each failure plane. Show locations of the cross sections used in slope stability analyses on the geotechnical map. Recommend mitigation if factors of safety are below County minimum standards.

7. The submitted report states that the design of the bank protection has been modified. Provide clarification, additional discussion, and details in regards to the stated design modification in relation to proposed bank protection shown on both the North and South banks of the Santa Clara River. The proposed Bank Stabilization is necessary for the feasibility of the proposed tract development and therefore must be addressed at the Tentative Map stage. Clarify design and provide additional information as necessary.

8. Show the following on the geotechnical map and Tentative Map:
   a. Final graded contours for the removal and recompaction of existing landslides.
   b. Delineate the location of the proposed disposal fill materials.
   c. Delineate the approximate limits of all soils unsuitable for the support of structure and prominently note/label with the note: "Unsuitable Soils - Not suitable for the Support of Structures".
   d. All recommended mitigation measures.

9. Requirements of the Geology Section are attached.

10. Include a copy of this review sheet with your response.

REGISTERED PROFESSIONAL ENGINEER YOSHIYA MORISAKU No. 67587 Exp. 6/30/11 CIVIL CALIFORNIA

2/5/10

Reviewed by _____ Date 2/5/09
Yosh

NOTICE: Public safety, relative to geotechnical subsurface explo[illegible] provided in accordance with current codes for excavations, inclusive of the Los Angeles County Code, Chapter 11.48, and the State of California, Title 8, Construction Safety Orders.
P:\Yosh\53108TentTk

Geotechnical Map

PREPARED FOR
Newhall Land and Farming Company

Project Location
Zone 1A Water Tank Site Grading
Castaic, California

SCALE	DRAWN BY	CHECKED BY
AS SHOWN	JH	
DATE	Figure 1	JOB NO.
2-23-2010		99-802-22

1329 SCOTT ROAD
BURBANK, CA. 91504
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FAX: (818) 531-1511
www.rtfranjian.com

RTF&A
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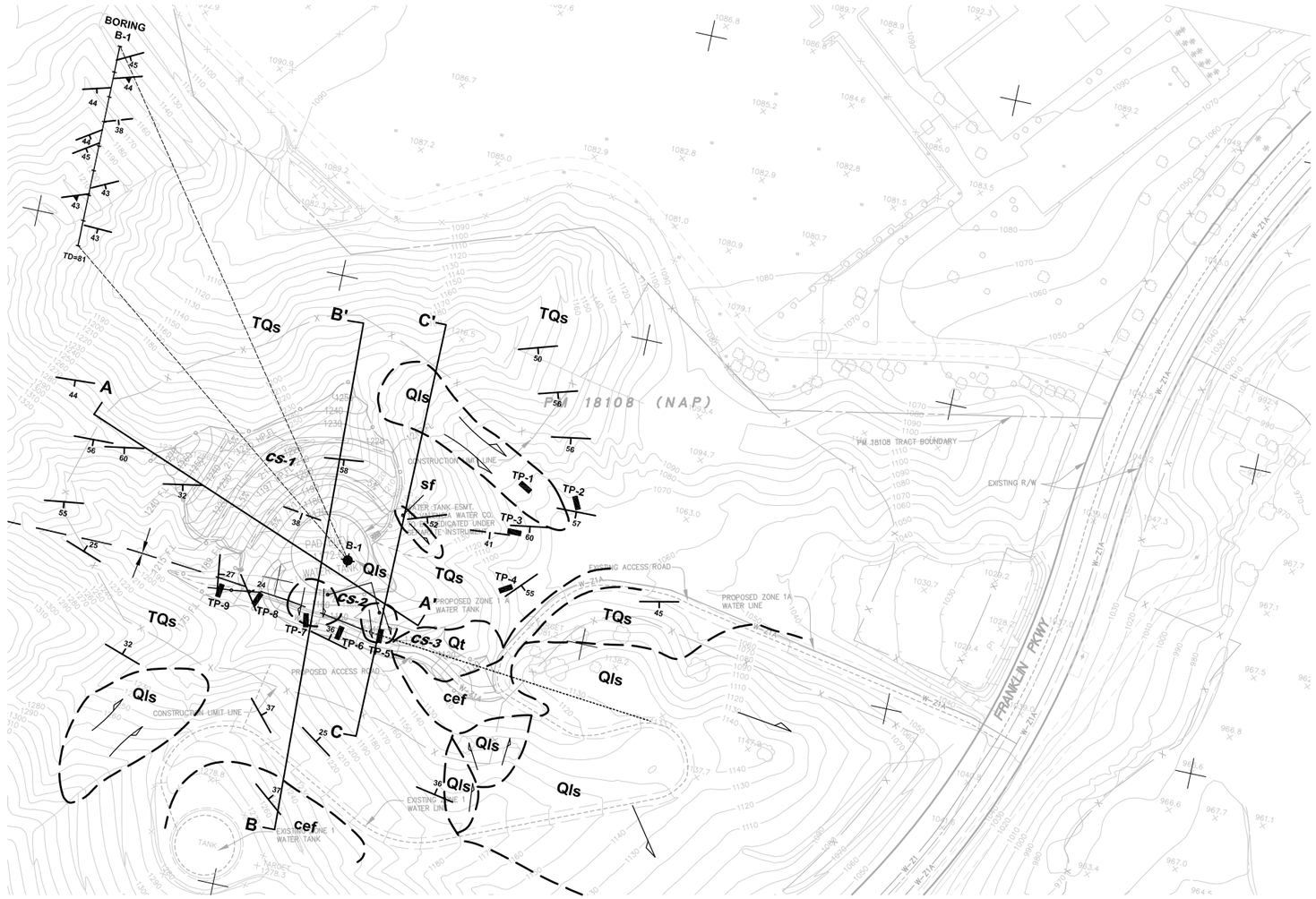


EXPLANATION

cef	Certified Engineered Fill
sf	Surficial Failure
Qls	Landslide Deposits
Qt	Terrace Deposits
TQs	Saugus Formation
	Geologic Contact, approximately located; queried where questionable
	Axial Trace of Syncline: dashed where approximate, dotted where concealed, queried where questionable; arrow on fold axis denotes direction of plunge
49	Strike and Dip of Bedding
41	Approximate Bedding
24	Strike and Dip of Bedding Shear
C — C'	Geologic Section Line
B-1	Boring Location and Number
TP-9	Test Pit Location and Number
CS-3	Cut Slope Designation
	Subdrain

OFFSITE ZONE 1A WATER TANK SITE	
RAW CUT	87,000 CUBIC YARDS
RAW FILL	2,000 CUBIC YARDS
EXPORT	87,000 CUBIC YARDS

NEWHALL RANCH

100' 50' 0' 100' 200'
GRAPHIC SCALE
Note: For reduced sized prints, original scale is in inches

PROPOSED OFFSITE
ZONE 1A WATER TANK GRADING
EXHIBIT

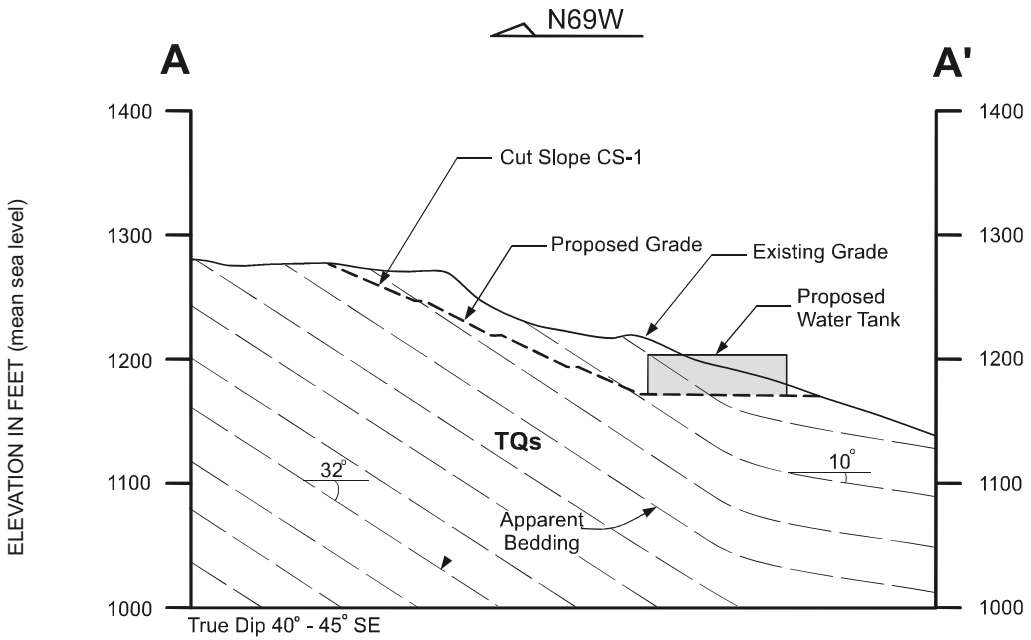
PSOMAS

DATE: 03-07-08 REVISED ON:
JOB No:1NRC010708

SHEET 1 OF 1
RV122-01

Plotted: 2/22/2010 10:37:41 AM :: Saved: 4/2/2008 9:46:34 AM :: C:\Documents and Settings\Administrator\My Documents\Work\Jobs\99-802\2-23-2010\RV122-01.dwg :: Administrator

GEOLOGIC SECTION A-A'



Geologic Section

PREPARED FOR
Newhall Land and Farming Company

PROJECT LOCATION
Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	JH	
DATE	FIGURE 2.1	JOB NO.
2-23-2010		99-802-22

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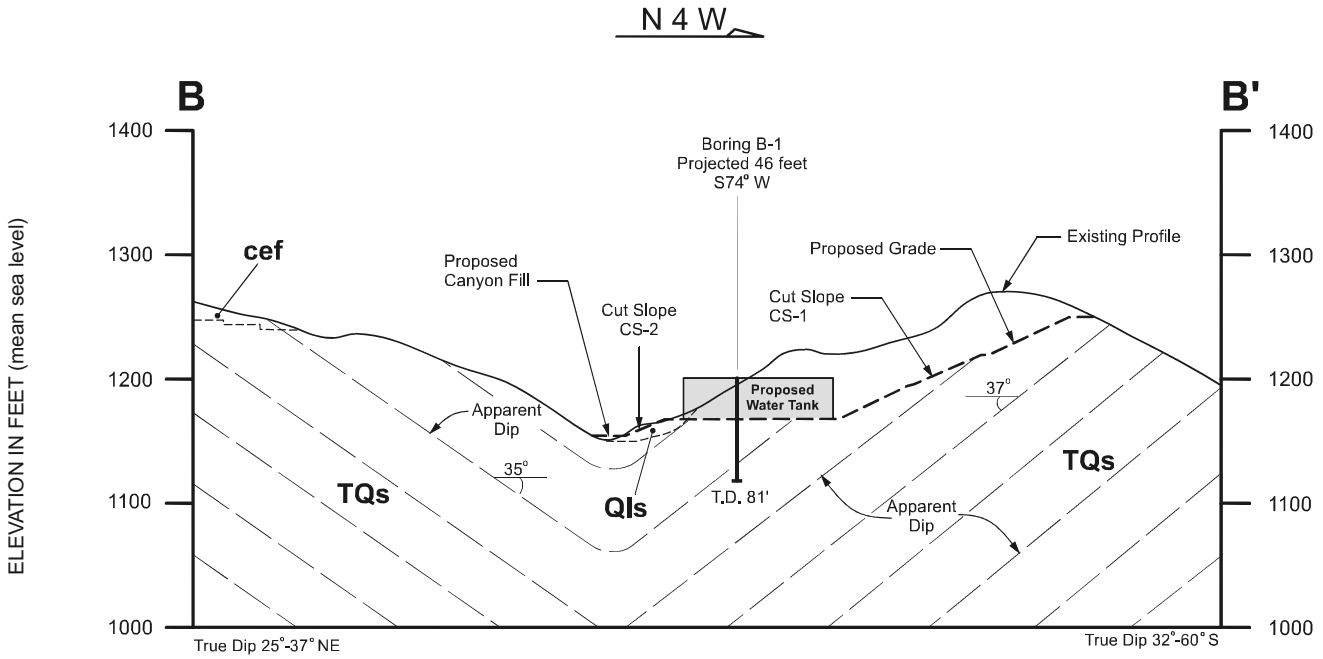
GEOLOGIC SECTION B-B'



Geologic Section

PREPARED FOR
Newhall Land and Farming Company

PROJECT LOCATION
Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	JH	
DATE	FIGURE 2.2	JOB NO.
2-23-2010		99-802-22

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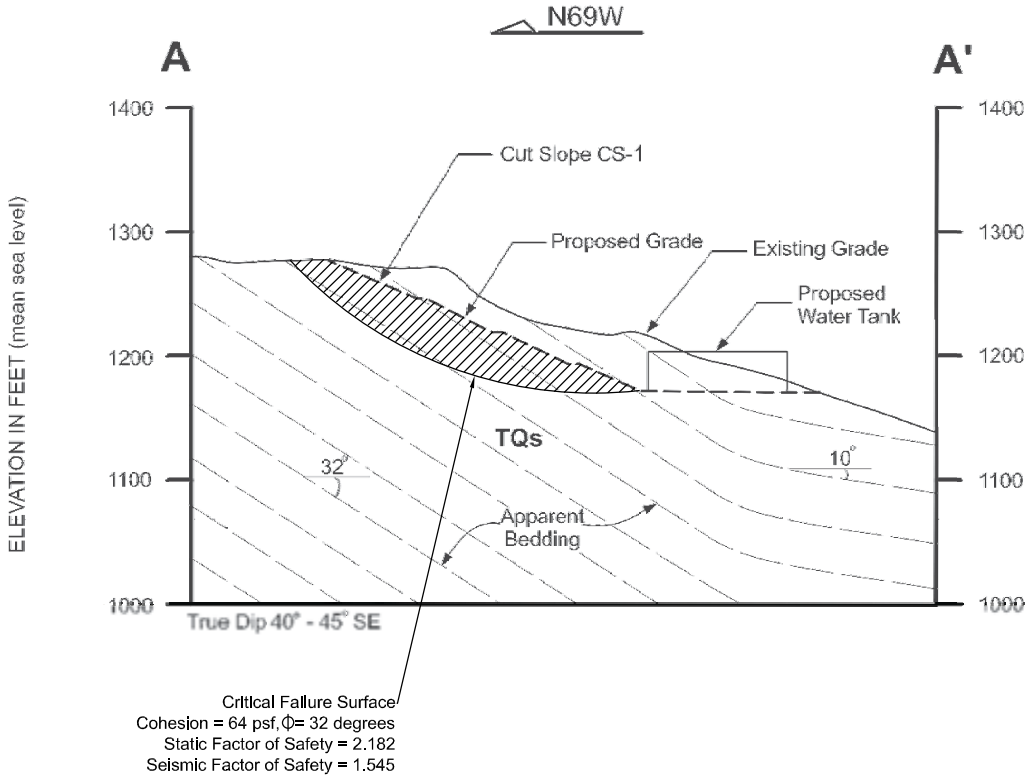
GEOTECHNICAL SECTION A-A'



Geotechnical Section

Newhall Land and Farming Company

Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53186
Castaic, California

SCALE: 1" = 100'

DRAWN BY: JH

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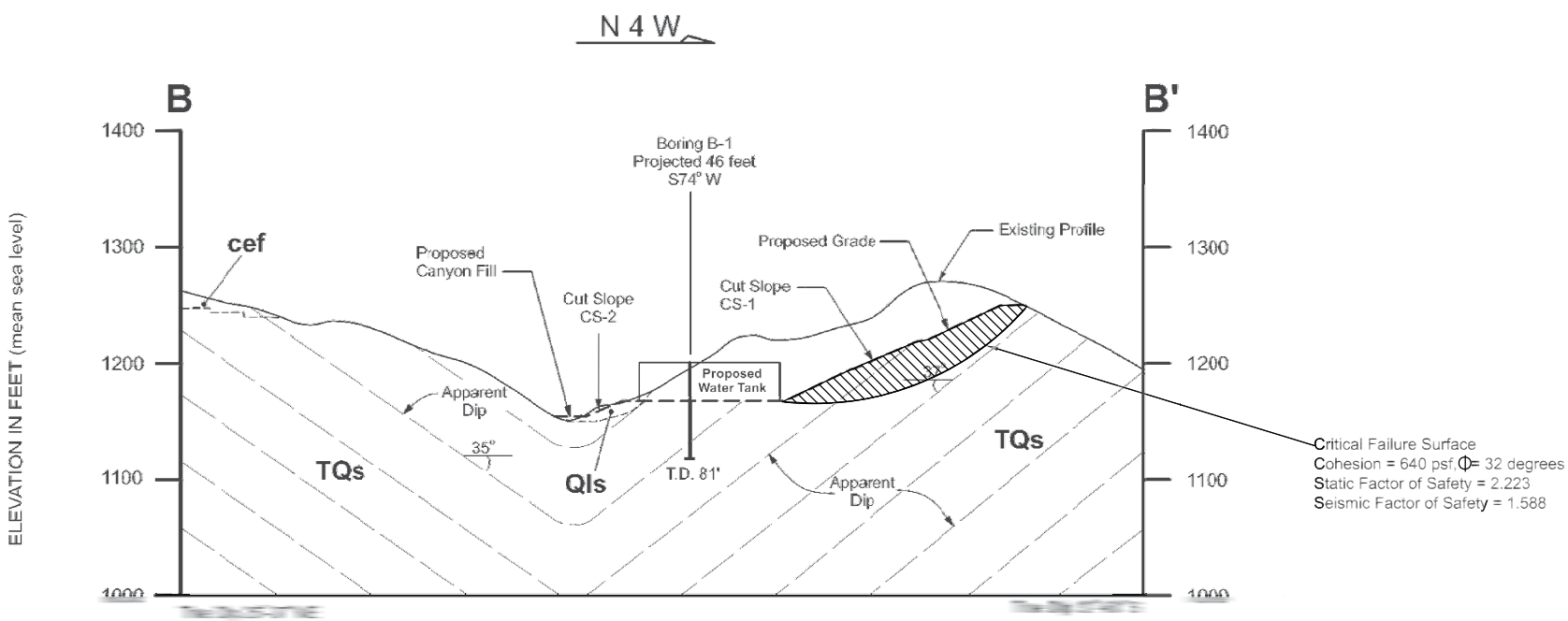
GEOTECHNICAL SECTION B-B'



Geotechnical Section

Newhall Land and Farming Company

Zone 1A Water Tank Site Grading
Vesting Tentative Tract Map No. 53108
Castaic, California

SCALE	DRAWN BY	CHECKED BY
1" = 100'	JH	
DATE	FIGURE 3.2	JOB NO.
2-23-2010		99-802-22

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Newhall Land and Farming Company
February 23, 2010
99-802-022

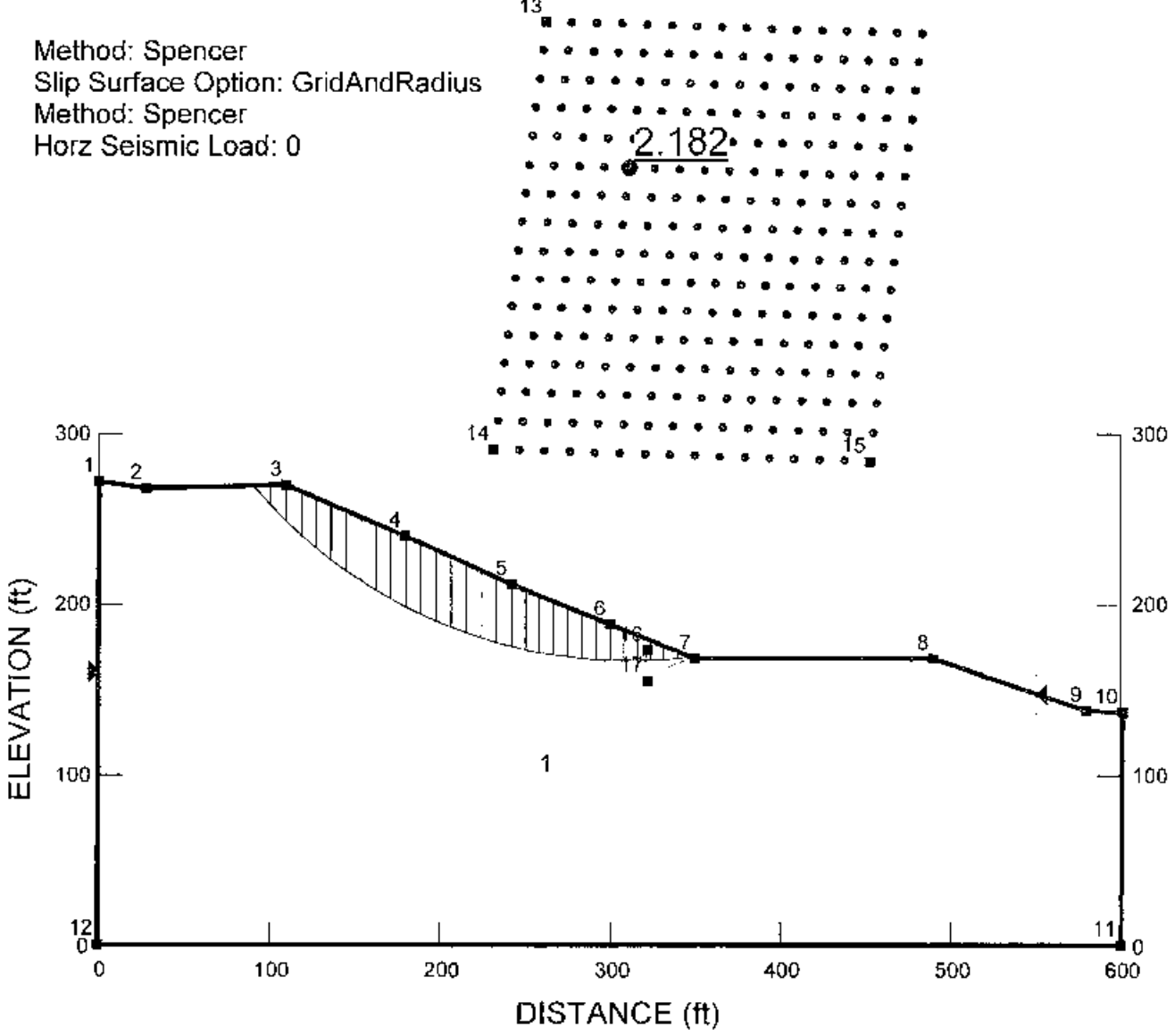
APPENDIX

SLOPE STABILITY CALCULATIONS

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

Title: Zone 1a Water Tank 99-802-22
Comments: TQs Cross Bedding
Name: Section A-A' STATIC.gsz
Directory: C:\Documents and Settings\Alan\My Documents\Reports\1999\99-802\slope stab
Date: 2/18/2010 Time: 1:57:13 PM
Material #: 1 Wt: 130 C: 640 Phi: 32 Model: MohrCoulomb

Method: Spencer
Slip Surface Option: GridAndRadius
Method: Spencer
Horz Seismic Load: 0



Title: Zone 1a Water Tank 99-802-22
Comments: TQs Cross Bedding
Name: Section A-A' Seismic.gsz
Directory: C:\Documents and Settings\Alan\My Documents\Reports\1999\99-802\slope stab
Date: 2/18/2010 Time: 2:00:33 PM
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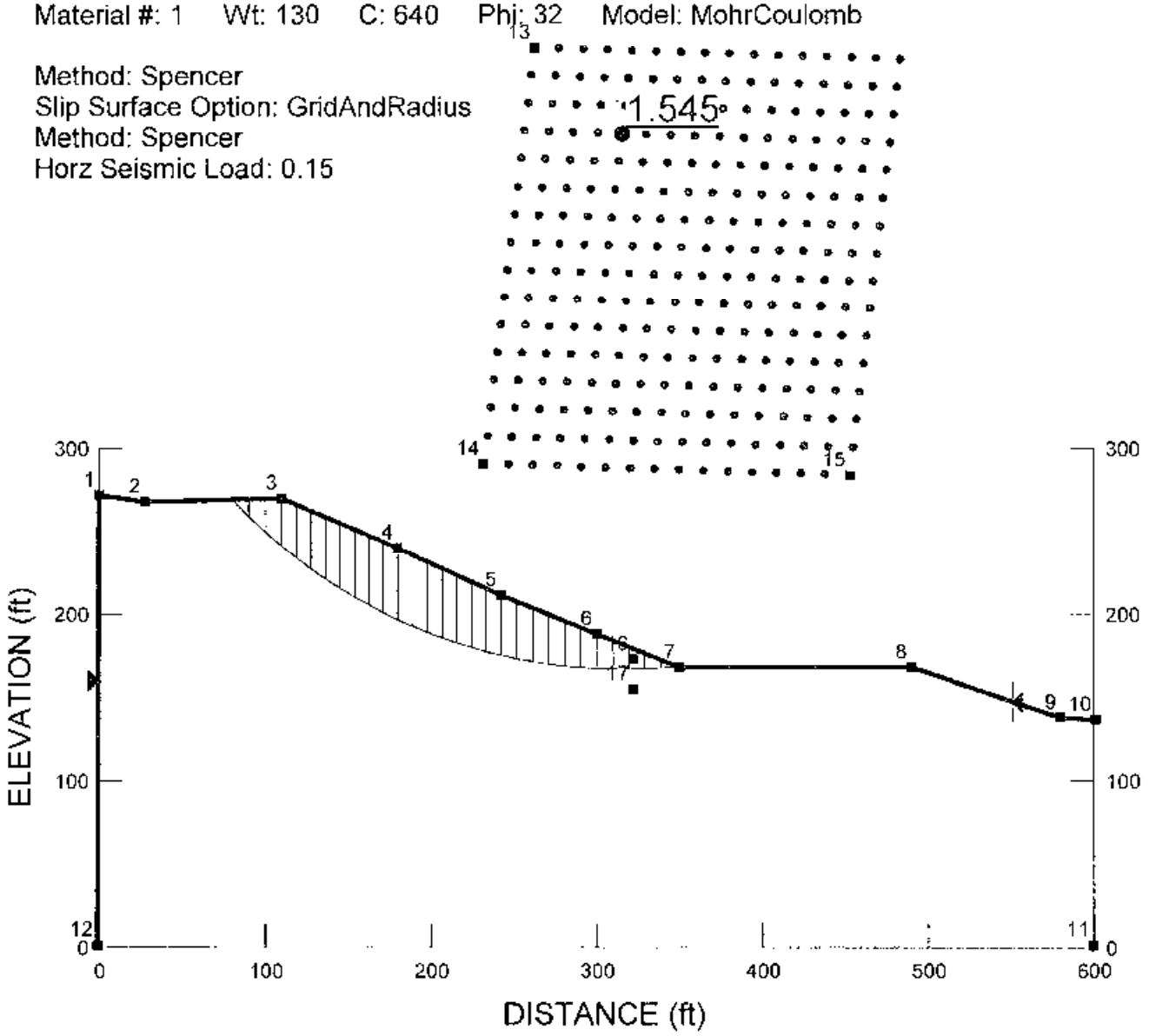
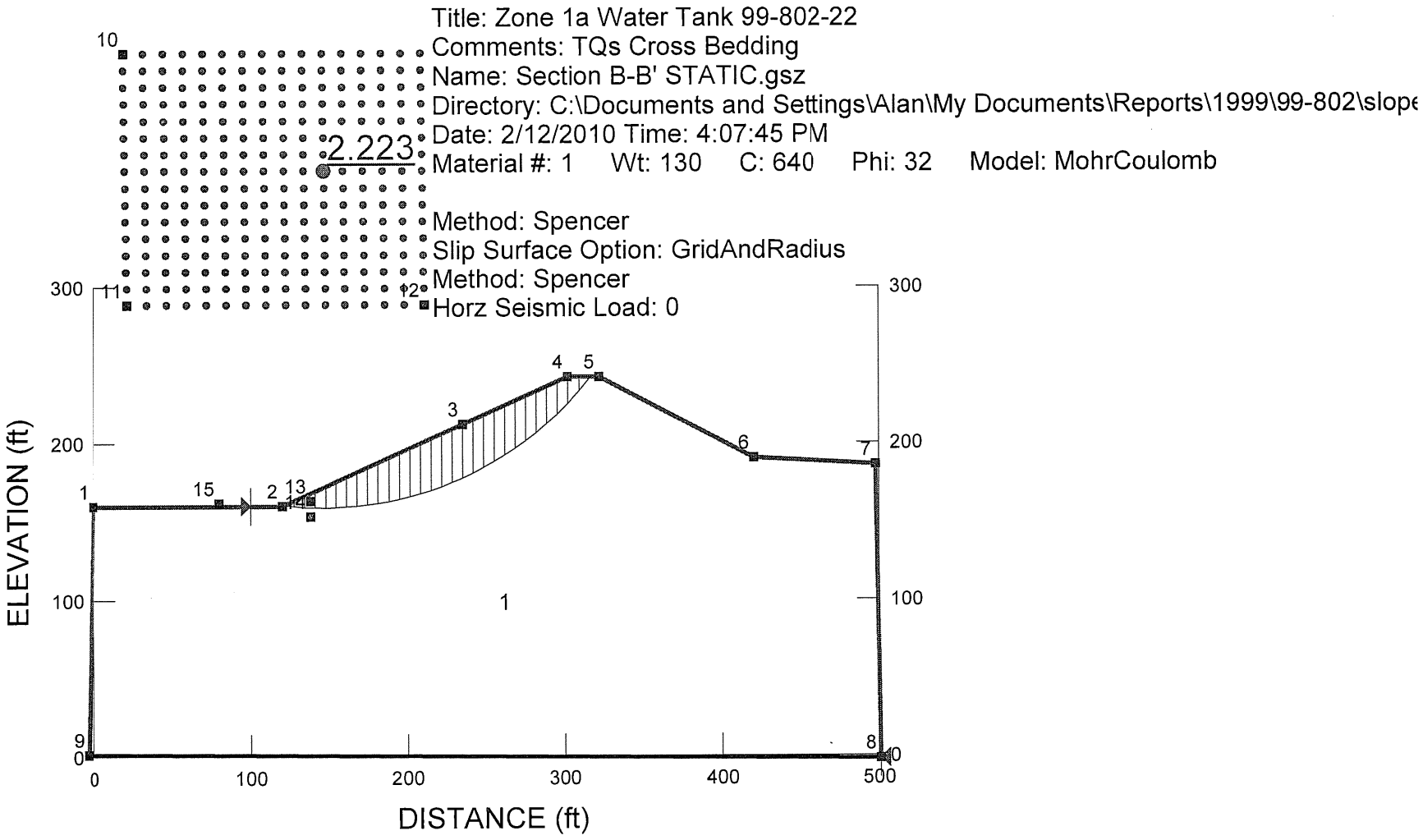
Method: Spencer
Slip Surface Option: GridAndRadius
Method: Spencer
Horz Seismic Load: 0.15



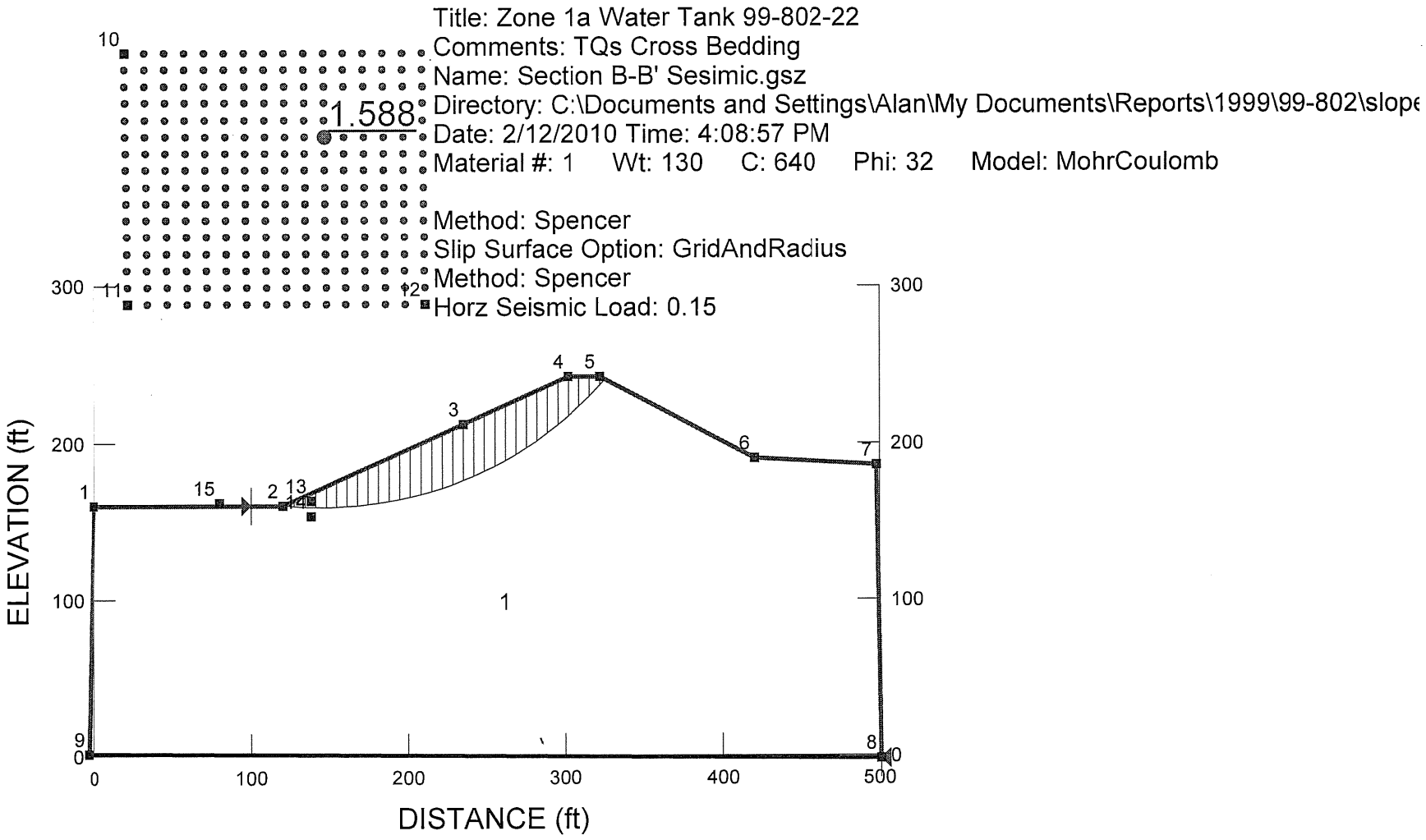
FOR USE WITH REPORT DATED 02/23/2010

RTF&A JOB NO. 99-802-022



FOR USE WITH REPORT DATED 02/23/2010

RTF&A JOB NO. 99-802-022



APPENDIX F4.2

Hydrology

Drainage Concept Report for Landmark Village Tentative Tract Map No. 53108, dated November 2009.

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS
LAND DEVELOPMENT DIVISION
STORM DRAIN AND HYDROLOGY SECTION

To: PACE AND PSOMAS

Attention MARK KREBS/PACE AND ERIK WINATA/PSOMAS

REVIEW OF TR NO. 53108/LANDMARK VILLAGE
TRANS DATE: 11/12/09 & 11/16/09 (VOLUME I BY PSOMAS); 11/09/09 (VOLUME II, III, IV, & V BY PACE)

[ ] Provide a drainage concept prior to approval of the tentative map. Sufficient information must be submitted to the Department showing the extent of the drainage problem and proposed solution.

[X] The Drainage Concept/SUSMP has been approved subject to conditions noted herein or shown on the returned map.

[ ] The drainage concept/SUSMP is unsatisfactory. Note the reasons stated herein or shown on returned map.

[ ] Prior to tentative map approval, comply with the Standard urban Stormwater Mitigation Plan (SUSMP) requirements. (www.888cleanla.com)

The first .75 inches of stormwater runoff volume from the site must be treated prior to discharging into stormwater conveyance systems.

[ ] Resubmit these sheets with check print, two (2) revised sets of the drainage concept, and the *MORA input/output file(s) on disk* (if applicable) for further consideration. Additional changes maybe required as determined by further review.

COMMENTS:

Submit a scanned copy of the approved plans and report for this project on a CD to Land Development Division prior to improvement plan submittal.

REVIEWED BY Yong Guo DATE 12/29/09

YONG GUO - (626) 458-4921

PSOMAS

DCR VOLUME I OF V

LANDMARK VILLAGE
TENTATIVE TRACT MAP 53108
DRAINAGE CONCEPT REPORT

November 2009

DRAINAGE CONCEPT/STORM WATER QUALITY PLAN

IS CONCEPTUALLY APPROVED

APPROVED BY: _____ RCE NO. 67363 DATE: 12/29/09

CHECKED BY: _____ DATE: 12/29/09

LAND DEVELOPMENT DIVISION

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

The DCR Volume I of V consists of three onsite drainage concept reports: Landmark Village/TR 53108 Drainage Concept, Offsite Borrow Areas Drainage Concept (Adobe Canyon borrow site and Chiquito Canyon borrow site-Under Conditional Used Permit), and Offsite Chiquito Landfill Drainage Concept.

Offsite hydraulics is addressed on the rest of DCR Volume.

TABLE OF CONTENT

TENTATIVE TRACT MAP
DRAINAGE CONCEPTREPORT
VOLUME I OF V

REPORT I : LANDMARK VILLAGE TENTATIVE TRACT MAP 53108 DRAINAGE CONCEPT REPORT-HYDROLOGY

REPORT II : OFF-SITE BORROW AREAS (UNDER CONDITIONAL USE PERMIT) DRAINAGE CONCEPT REPORT

REPORT III : OFF-SITE CHIQUITO LANDFILL DRAINAGE CONCEPT REPORT

REPORT I

LANDMARK VILLAGE
TENTATIVE TRACT MAP 53108
DRAINAGE CONCEPT REPORT-HYDROLOGY

LANDMARK VILLAGE
TENTATIVE TRACT MAP 53108
DRAINAGE CONCEPT REPORT
HYDROLOGY

Psomas Project No: 1NRC010708
Prepared: November 2009

Prepared for:

NEWHALL LAND
23823 W. Valencia Blvd.
Valencia, CA 91355-2194
Telephone: (661) 255-4000

Prepared by:

PSOMAS
28480 Avenue Stanford, Suite 200
Santa Clarita, California 91355
Telephone: (661) 219-6000
Fax: (661) 775-2718

11/12/09

ERIK WINATA R.C.E. No. C67285 Exp. Date **09-30-10**

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Purpose and Scope

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Watershed Characteristics
Existing Condition Results
Proposed Condition Results
Baseline Hydrology Comparison

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4. Limitations **18**

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APPENDICES

1. **LACDPW Hydrology Data**
2. **Existing Condition TC Calculations**
3. **Proposed Condition TC Calculations**
4. **Existing Condition LACDPW MORA Calculations**
5. **Existing Conditions – Burned LACDPW MORA Calculations**
6. **Proposed Condition LACDPW MORA Calculations**
7. **Proposed Condition – Burned LACDPW MORA Calculations**
8. **LA County Standard SUSMP Calculations**
9. **Geosyntec Memorandum – Stormwater Control BMP Sizing**
10. **Drainage Concept Exhibits**

1.0 Introduction

NOTE: This Drainage Report was conceptually approved by Land Development Division, Los Angeles County Department of Public Work on July 5, 2007. A revised drainage report is needed for consistency with revised Vesting Tentative Tract Map.

Project Background

Psomas has been retained by the Newhall Land Co. to prepare a drainage concept report in conjunction with the preparation of Tentative Tract Map No. 53108 for Landmark Village. The project site, located in an unincorporated portion of Los Angeles County, is bounded by the Santa Clara River on the south, the SR 126 on the north and CA Interstate 5 on the east, consists of single- a -family residential, commercial, and recreational uses. Landmark Village is a part of Newhall Land's Newhall Ranch, an approximately 12,000-acre proposed master planned community. Two borrow site areas are associated with Landmark Village but are under a Conditional Use Permit (CUP), not part of the tentative tract map itself, and are addressed in Report II and Report III of Landmark Drainage Concept Report Volume I.

Purpose and Scope

The project falls under the jurisdiction of the Los Angeles County Department of Public Works (LACDPW). The purpose of this drainage concept report is:

- To meet Los Angeles County Land Development requirements in support of the tentative tract map submittal, allowing final design and construction to proceed in a timely manner;
- To determine the proposed development's impact (increase peak flow rates) to existing hydrologic conditions;
- To provide sufficient detailed hydrology information to support detailed hydraulic design of storm drainage facilities; and
- To document that the Los Angeles County Standard Urban Stormwater Mitigation Plan (SUSMP) requirements will be met.

It should be noted that detailed catch basin, storm drain and detention basin hydraulics analysis and sizing is beyond the scope of this drainage concept report.

The following figure is a vicinity map of the Landmark Village site.

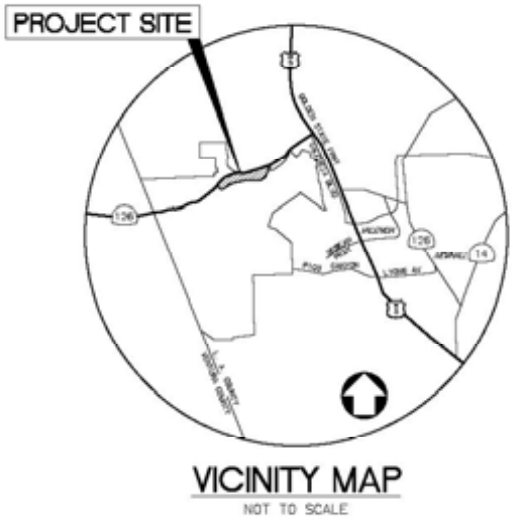


Figure 1. Vicinity Map

2.0 Hydrology

General Approach

The watershed of the project was divided into several sub-basins that were identified and characterized for both existing and proposed conditions. Computer modeling was used to estimate the runoff for the 50-year storm events; the analysis considered burned hydrology but no additional bulking factors were used in the proposed conditions, as sediment trapping devices will be installed upstream of the project. In addition, the need for water quality mitigation (SUSMP) was evaluated and is addressed in section 4.

Data Sources

The primary sources of data were the *LACDPW Hydrology/ Sedimentation Manual and Appendices* (LACDPW 1991, 1992, 1993, 2002) and the LA County *Standard Urban Stormwater Mitigation Plan* (March 2000). Other key sources included the *TC (v1.0),* and *MORA (v7.2)* manuals and the Newhall Ranch Specific Plan Master Hydrology and Drainage Concept, Sikand December, 1999 (appended for reference).

Watershed Characteristics

The project site is bounded by the Santa Clara River on the south, SR 126 on the north, and CA Interstate 5 on the east. The off-site contributing drainage area for the site encompasses approximately 236 acres and extends predominantly in the northerly direction from the site. The majority of the area is undeveloped land with moderate slopes. The Chiquito Canyon Landfill drainage area is addressed in a separate study and excluded from this analysis.

The soil of the watershed is classified by Los Angeles County to be Type 020 and 097, as shown on the figure (LACDPW 1-H1.43) included in Appendix 1. The figure also shows that the drainage area is associated Debris Potential Area (DPA) number 9. As previously discussed, the proposed uses of sediment trapping devices preclude the necessity of bulking factors in the proposed conditions. The peak bulking factor curves and peak debris production rates used in the analysis are included in Appendix 1 (LACDPW P2 & P5).

Landmark Village is located in Los Angeles County's Rainfall Isohyet 5.8, as shown on the aforementioned figure in Appendix 1.

The drainage area for the site consists of six sub-basins that independently drain toward the Santa Clara River; refer to the Existing Condition Hydrology Map included with this report. The sub-basins drain southwesterly on site, ultimately tributary to the Santa Clara River.

The *LACDPW TC (TC_calc_depth.xls, June2002)* program was used to calculate the time of concentration for each sub area. The *LACDPW MORA (v7.2)* computer program was used for the *Modified Rational Method* calculations and calculation of peak runoff rates and hydrographs. The TC calculations and MORA program output are provided in Appendix 2-3 and 4-5, respectively. In accordance with LACDPW requirements, the 50-year burned, bulked storm event was used as the main design storm in the analysis.

Ex ti tion Results

Existing condition hydrology results for the Capital Flood storm event is summarized in Table 1 below.

Table 1: Existing Condition Hydrology Summary

Sub basins	Area (Ac)	50-Year Storm Event[A]			
		Time of Conc. (min)	Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb} (cfs) 1.27*Q_b
100A	32.7	22	30	41	52
110A	49.6	20	42	59	75
Cumulative	**82.3**			**88**	**112**
200A	17.3	17	17	23	29
210A	35.8	24	28	39	50
Cumulative	**53.1**			**60**	**76**
400B	18.4	24	14	20	25
405B	38.9	28	27	39	50
408C	15.3	8	24	31	39
410C	44.3	19	41	56	71
415B	35.3	11	47	62	79
420A	34.4	24	26	37	47
425A	39.9	20	35	48	61

Sub basins	Area (Ac)	50-Year Storm Event[A]			
		Time of Conc. (min)	Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb} (cfs) 1.27*Q_b
Cumulative	**226.5**			**239**	**304**
500A	26.5	20	23	32	41
510A	40.0	24	31	44	53
Cumulative	**66.5**			**64**	**81**
CQT-1A	6.1	8	10	13	16
CQT-2A	3.6	6	8	10	13
CQT-3A	1.8	5	4	5	6
CQT-4A	12.3	10	17	22	28
CQT-5A	4.4	5	9	11	14
CQT-6A	24.9	15	27	37	47
CQT-7A	2.1	5	4	5	6
CQT-8A	2.8	5	7	8	10
CQT-9A	31.8	14	36	49	62
CQT-10A	15.6	11	21	28	36
CQT-11A	10.2	17	16	17	22
CQT-12A	11.7	10	25	27	34
620A	12.4	22	10	14	18
Cumulative	**140**			**160**	**203**
Total	**568.1**			**611**	**776**

Notes

A. *Burned and bulked flow. The Cumulative flow is the result shown in the MORA tabulations that are in the Appendix. The Total Flow is the sum of the Cumulative Flows shown.*

Post-Project Condition Results

The post-project condition hydrologic analysis was based on the preliminary concept drainage plan prepared by Psomas, included with this report. The post-project (developed) Condition Drainage Concept Map is also included with this report. Post-development drainage patterns do not significantly differ from the aforementioned existing characteristics.

As with the proposed condition analysis, the *LACDPW TC* program was used to calculate the time of concentration, and the *LACDPW MORA* computer program was used to calculate the peak runoff rates and hydrographs for landmark offsite. The calculations and program output are provided in Appendix 3 and 5 respectively.

Land use impervious factors used were based on combination *LACDPW Hydrology/Sedimentation Appendix* and case by case studies. Details for land use impervious factors are shown on Appendix I.

Post-project condition hydrology results for the 50-year storm event is summarized in Table 3.

Table 3: Post-Project Condition Hydrology Summary

Sub basins	Area(Ac)	Time of Conc. (min)	50-Year Storm EventA			
			Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb}(cfs) 1.27*Q_b	Qdesign (cfs) (MORA)
R -1A	18	19	22	26	33	26
R -2A	39	22	43	52	66	52
R -3B	15	7	32	37	47	37
R -4B	44	15	61	73	93	73
R -6A	35	9	65	76	97	76
R -6A (Retention)	-	-	-	-	-	-90
R -7A	16	29	23			23
Subtotal(Off-Site)	**168**			**ΣQ_{b+d}-detention**		**98**
RVE-8A	6.35	8	15.8			15.8
RVE-9B	7.04	10	15.8			15.8
RVE-10B	0.69	6	2.0			2.0
RVE-11C	1.02	5	3.2			3.2
RVE-12C	0.63	5	2.0			2.0
RVE-13C	0.87	5	2.7			2.7
RVE-14C	0.2	5	0.6			0.6
RVE-15C	0.54	10	1.2			1.2
RVE-16C	2.78	10	6.3			6.3
RVE-17C	5.17	8	12.9			12.9
RVE-18C	0.66	11	1.4			1.4
RVE-19C	0.3	7	0.8			0.8
RVE-20C	4.68	18	8.0			8.0
RVE-21D	1.82	7	4.8			4.8
RVE-22D	0.34	7	0.9			0.9
RVE-22.1D	3.56	7	9.5			9.5
RVE-23D	0.55	10	1.2			1.2
RVE-24D	0.77	12	1.6			1.6
RVE-25D	0.21	5	0.7			0.7
RVE-26D	5.09	11	11.0			11.0

Sub basins	Area(Ac)	Time of Conc. (min)	50-Year Storm EventA			
			Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb}(cfs) 1.27*Q_b	Qdesign (cfs) (MORA)
RVE-27D	0.95	13	1.9			1.9
RVE-28D	1.78	11	3.8			3.8
RVE-29D	2.08	11	4.5			4.5
RVE-30E	0.53	9	1.3			1.3
RVE-31E	0.47	5	1.5			1.5
RVE-32E	0.33	5	1.0			1.0
RVE-33E	5.86	9	13.9			13.9
RVE-34E	5.29	7	14.0			14.0
RVE-35E	1.23	8	3.1			3.1
RVE-36E	0.68	10	1.5			1.5
RVE-37E	1.7	10	3.8			3.8
RVE-38F	0.25	6	0.7			0.7
RVE-39F	2.03	6	5.8			5.8
RVE-40F	0.05	5	0.2			0.2
RVE-41F	1.79	9	4.2			4.2
RVE-42F	0.5	9	1.2			1.2
RVE-43F	3.86	12	8.0			8.0
RVE-44F	2.86	11	6.2			6.2
RVE-45F	0.14	5	0.4			0.4
RVE-46F	2.69	13	5.4			5.4
RVE-51G	2.99	8	7.5			7.5
RVE-52G	1.24	7	3.3			3.3
RVE-53G	1.21	6	3.5			3.5
RVE-54G	1.19	6	3.4			3.4
RVE-55G	0.82	5	2.6			2.6
RVE-56G	1.02	7	2.7			2.7
RVE-57G	3.37	9	8.0			8.0
RVE-58G	1.93	12	4.0			4.0
RVE-59G	1.9	10	4.3			4.3
RVE-60G	9.88	11	21.3			21.3
RVE-61G	1.36	9	3.2			3.2
RVE-62G	1.22	8	3.0			3.0
RVE-63G	4.11	10	9.3			9.3
Subtotal(RVE)	**110.58**					-

Sub basins	Area(Ac)	Time of Conc. (min)	50-Year Storm EventA			
			Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb}(cfs) 1.27*Q_b	Qdesign (cfs) (MORA)
RVC-1A	11.66	12	24.0			24.0
RVC-2A	4.08	9	9.7			9.7
RVC-3A	2.39	14	4.6			4.6
RVC-4A	1.67	13	3.3			3.3
RVC-5A	0.4	5	1.3			1.3
RVC-6A	0.78	5	2.4			2.4
RVC-7A	0.55	5	1.7			1.7
RVC-8A	1.52	7	4.0			4.0
RVC-9B	8.39	11	18.1			18.1
RVC-10B	1.03	9	2.4			2.4
RVC-11B	0.74	8	1.8			1.8
RVC-12B	9.79	10	22.0			22.0
RVC-13B	1.34	11	2.9			2.9
RVC-14B	2.83	11	6.1			6.1
RVC-15C	3.54	8	8.8			8.8
Subtotal(R)	**50.71**					-
RVW-1A	0.8	6.0	2.4			2.4
RVW-2A	3.2	5.0	10.0			10.0
RVW-3A	1.5	7.0	3.9			3.9
RVW-4A	3.3	8.0	8.3			8.3
RVW-5B	0.3	5.0	1.1			1.1
RVW-6B	1.2	5.0	3.8			3.8
RVW-7B	0.2	5.0	0.6			0.6
RVW-8B	0.2	5.0	0.6			0.6
RVW-9B	2.2	7.0	5.9			5.9
RVW-10B	1.3	7.0	3.5			3.5
RVW-11B	0.3	5.0	1.1			1.1
RVW-12B	3.5	15.0	6.6			6.6
RVW-12C	0.9	12.0	1.8			1.8
RVW-13C	3.6	8.0	9.0			9.0
RVW-14C	2.6	12.0	5.3			5.3
RVW-15C	2.1	9.0	5.0			5.0
RVW-16C	1.0	7.0	2.6			2.6

Sub basins	Area(Ac)	Time of Conc. (min)	50-Year Storm EventA			
			Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb}(cfs) 1.27*Q_b	Qdesign (cfs) (MORA)
RVW-17C	1.9	11.0	4.1			4.1
RVW-18D	1.9	7.0	5.1			5.1
RVW-19D	0.5	6.0	1.5			1.5
RVW-20D	0.4	6.0	1.1			1.1
RVW-21D	11.0	15.0	20.4			20.4
RVW-22D	1.9	13.0	3.7			3.7
RVW-23D	1.3	12.0	2.6			2.6
RVW-24D	4.9	11.0	10.6			10.6
RVW-25D	0.5	6.0	1.4			1.4
RVW-26D	1.1	5.0	3.5			3.5
RVW-26.1D	0.6	5.0	1.7			1.7
RVW-27D	1.0	6.0	2.9			2.9
RVW-28D	17.7	11.0	38.1			38.1
RVW-29D	4.1	8.0	10.1			10.1
RVW-31E	2.2	10.0	4.9			4.9
RVW-32E	0.7	5.0	2.2			2.2
RVW-33E	2.0	7.0	5.3			5.3
RVW-34E	2.4	10.0	5.5			5.5
RVW-35E	0.7	7.0	1.7			1.7
RVW-36E	0.3	5.0	0.8			0.8
RVW-37E	0.4	5.0	1.2			1.2
RVW-38E	0.8	6.0	2.2			2.2
RVW-39E	5.7	11.0	12.3			12.3
RVW-39.1E	0.8	8.0	1.9			1.9
RVW-40E	7.4	10.0	16.7			16.7
RVW-41E	2.3	10.0	5.2			5.2
RVW-42E	1.0	7.0	2.8			2.8
RVW-43G	1.3	8.0	3.2			3.2
RVW-44G	1.9	5.0	6.0			6.0
Subtotal(R)	**107.0**					**-**
		ΣQ_{peak} (Line 1 through Line 15)				**515**
CQT-1A	30.0	11	42	54	69	69
CQT-2A	6.0	6	12	15	19	19
CQT-3B	4.0	5	9	11	14	11
CQT-4B	20.0	14	24	31	39	31

Sub basins	Area(Ac)	Time of Conc. (min)	50-Year Storm EventA			
			Q_u (cfs)	Q_b (cfs) (MORA)	Q_{bb}(cfs) 1.27*Q_b	Qdesign (cfs) (MORA)
CQT-5B	7.0	7	13	16	20	16
CQT-6B	6.0	7	11	14	18	14
CQT-7B	13.0	9	24	28	35	28
CQT-9C	8.0	8	13	17	22	17
CQT-10C	9.0	11	12	16	20	16
CQT-11C	12.0	18	20			20
CQT-12C	2.0	16	2			2
CQT-13C	8.0	22	11			11
Subtotal (CQT) (offsite)	**125**	ΣQ_{peak} (CQT offsite)				**159**
SUM	**560**					**772[A]**

Notes

A. The Cumulative flow is the result shown in the MORA tabulations that are in the Appendix. The Total Flow is the sum of the Cumulative Flows shown. Burned flow for Subareas RVE 1A thru 6A, Develop flow for the remaining Subareas RVE, S -3B, CQT-4B, CQT-5B, CQT-9C Burned and bulked flow for Subareas CQT-1A and, CQT-2A, Develop flow for Subareas CQT-7B, CQT-11C , CQT-12C and CQT-13C.

Baseli

In accordance with Los Angeles County hydrology method, existing condition burned and bulked flow is assessed with proposed condition flow burned, unbulked due to the proposed addition of debris facilities where needed.

Table 4: Existing vs. Proposed Condition Hydrology Comparison Summary

50-Year Storm Event						
Sub area	**Area (Ac)**			**Q_{50} (cfs)**		
	Exist.[A]	**Prop.[B]**	**Diff.**	**Exist[C].**	**Prop.[D]**	**Diff.**
Total Study Area	**568**	**560**	**-8**	**776**	**772**	**-4**

Notes

A. *Existing Tributary Drainage Area*
B. *Proposed Tributary Drainage Area.*
C. *Burned and bulked flow*
D. *Burned flow for Subareas RVE 1A thru 6A, Develop flow for the remaining Subareas RVE, S , Burned flow for Subareas CQT-3B, CQT-4B, CQT-5B, CQT-9C Burned and bulked flow for Subareas CQT-1A and, CQT-2A, Develop flow for Subareas CQT-7B, CQT-11C , CQT-12C and CQT-13C.*

3.0 Standard Urban Stormwater Mitigation Plan (SUSMP)

Structural or Treatment Control Best Management Practices (BMPs) a
).

With respect to BMP sizing for Newhall Ranch projects, a performance standard has been established via another water quality report (Geosyntec Landmark Water Quality Technical Report, February 2008) that covers the entire Ranch (the Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan). That plan states that all BMPs on the ranch will be sized to capture and treat 80 percent of the average annual runoff volume. The document further states that all BMPs will be sized to achieve this standard or the Los Angeles County SUSMP Manual sizing standard, whichever is more stringent.

Hence, all BMPs within Landmark Village project will be sized to achieve this Sub-Regional Stormwater Mitigation Plan or the Los Angeles County SUSMP Manual sizing standard, whichever is more stringent

Based on LA county methodology, minimum SUSMP requirements have been calculated and shown on Appendix 6.
Based on Geosyntec Water Quality Technical Report methodology, minimum SUSMP requirements have been calculated and shown on Appendix 7 (Geosyntec memorandum Table 1, 2 &3).

The LACDPW intensity-duration data and the runoff coefficient curve for soil type 20 are included in Appendix 1. Drainage area design parameters are shown on Appendix 3.

The BMPs requirements are summarized in the table 5. Refer to SUSMP Map (Sheet 2A) on Appendix 8 for BMPs locations and contributing subareas.

Table 5: SUSMP Requirements

Prop. BMP Site Number	Subarea	Area (ac)	LA County Required Design Flow(Qpm) cfs	Geosyntec Newhall standarad Design Flow(Qpm) cfs	LA County Required Mitigated Volume (Vm) Acre-Ft	Geosyntec Newhall standarad Mitigated Volume (Vm) Acre-Ft	Qpm or Vpm Designed
BMP 1	RVE-9B/20C	11.72			0.35	0.39	0.39 Ac.ft
BMP 2	RVE-11C/12C/13C/14C/15C/16C/17C/18C/19C/29D	14.25	-	-	0.53	0.58	0.58 Ac.f
BMP 3	RVE-21D/22D/22.1D/23D/24D/25D/26D/27D/28D/37E	16.77	-	-	0.70	0.74	0.74 Ac.ft
BMP 4	RVE-30E/31E/32E/33E/34E/35E/36E/46F	17.07	-	-	0.66	0.71	0.71 Ac.f
BMP 5	RVE-38F/39F/40F/41F/42F/43F/44F/45F/8A	13.00	-	-	0.50	0.54	0.54 Ac.ft
BMP 6/7	RVC-1A/2A/3A/4A/5A/6A/7A/9B/10B/11B/12B/13B/14B/ RVW-1A/2A/3A/4A/5B/6B/7B/8B/9B/10B/11B/12B/17C RVW-12/C/13C/14C/15C/16C/43G	77.23	-	-	3.24	3.45	3.45Ac.f
BMP 8	RVW-18D/19D/20D/21D/22D/23D/24D/25D/26D/26.1D//27D/28D/29D/31E/32E/33E/34E/35E/36E/44G	56.90	-	-	2.55	4.6	4.6 Ac.ft
BMP 9	RVW-37E/38E/40E/42E	9.59	-	-	0.45	0.47	0.47 Ac.f
BMP 10	RVW-39.1E/39E/41E	8.50	-	-	0.30	0.34	0.34 Ac.f

Prop. BMP Site Number	Subarea	Area (ac)	LA County Required Design Flow(Qpm) cfs	Geosyntec Newhall standarad Design Flow(Qpm) cfs	LA County Required Mitigated Volume (Vm) Acre-Ft	Geosyntec Newhall standarad Mitigated Volume (Vm) Acre-Ft	Qpm or Vpm Designed
BMP 11/12	RVE-51G/52G/53G/54G/55G/56G/57G/58G/59G/63G	19.78	-	-	0.79	0.84	0.84 Ac.ft
BMP 13	RVE-15C	3.54	0.15	0.24	-	-	0.24 cfs
BMP 14	RVE-60G	9.88	-	-	0.43	0.45	0.45 Ac.f
BMP 15	RVE-62G	1.22	0.20	0.31	-		0.31 cfs
BMP 16	RVE-61G	1.36	0.22	0.34	-		0.34 cfs

**1 - SUSMP for Offsite development within Caltran R/W to be address per separate Caltran SR126 Design(RVE7A and CQT11C-13C).*

Based on Qpm or Vpm designed on Table 5, treatment goals can be achieved through a variety of BMPs including detention basins, biofilter swales, catch basin inserts and media filtration. Type of BMPs design will comply with both LA County SUSMP Manual and Geosyntec Water Quality Technical Report (February 2008).

Landmark Village Project will consist of one extended Bio (ED) basin, eleven Bio Basin areas, one vegetated swale, and two modular wetlands. The ED basin, identified as BMP-8, will provide treatment for 57 acres (21%) of the project development area; the 11 Bio Basin areas will provide treatment for 204 acres 560 of the project development area; the vegetated swale will provide treatment for 3.54 acres (~1.5%) of the project development area; and two modular wetlands will provide treatment for 2.6 acres (~1%) of the project development area.

Table 5 Summarized BMPs type for each BMP site. It also illustrates how the required treatment capacity can be met based on the area allocated.

Table 5: BMPs Sites & Design

Prop. BMP Site Number	Location of Prop. BMP Site	BMP Type	Required Qpm or Vpm (Table 5)	Area provided for SUSMP Treatment (AC)	Avg Width	Avg Length	Area provided for SUSMP Treatment (AC)	Max Volume With Vertical Wall	Max Volume With 2:1 Side Slope, 1' freeboard , 2.5' Total Depth
BMP 1	RVE-20C	Bio Basin	0.39 Ac.ft	1.18	30	1650	1.18	1.77	1.36
BMP 2	RVE-29D	Bio Basin	0.58 Ac.ft	0.63	30	830	0.63	0.95	0.92
BMP 3	RVE-37E	Bio Basin	0.74 Ac.ft	0.54	45	580	0.54	0.81	0.72
BMP 4	RVE-46F	Bio Basin	0.71 Ac.ft	1.08	40	1120	1.08	1.62	1.34
BMP 5	RVC-8A	Bio Basin	0.54 Ac.f	.41	45	395	0.41	0.62	0.56
BMP 6/7	RVW-12B/RVW-17C/43G	Bio Basin	3.45Ac.ft	2.73	45	2550	2.73	4.09	3.45
BMP 8	RVW-44G	WQ Extended Bio Basin *b	4.6 Ac.ft	-	-	-	-	5.06 *a	-
BMP 9	RVW-42E	Bio Basin	0.47 Ac.f	.48	45	360	0.48	0.72	0.42
BMP 10	RVW-41E	Bio Basin	0.34 Ac.ft	0.68	45	630	0.68	1.02	1.14
BMP 11/12	RVE-59G/63G	Bio Basin	0.84 Ac.ft	0.93	30 & 15	680 &1000	0.93	1.40	0.86
BMP 13	RVE-15C	Vegatative Swale	0.24 cfs	-	18	330	-	-	-
BMP 14	RVE-60G	Bio Basin	0.45 Ac.ft	0.3	-	-	0.30	0.45	0.51
BMP 15	RVE-62G	Modular Wetland Unit	0.31 cfs	-	-	-	-	-	-
BMP 16	RVE-61G	Modular Wetland Unit	0.34 cfs	-	-	-	-	-	-
BMP 17	RVE-8A	Bio Basin	0.13 Ac.ft	0.2	-	-	0.20	0.30	0.29

** - Volume based on 3:1 side slope, 2 ft freeboard. As shown on Extended Biodetention Basin Cross Section on SUSMP Map(Sheet 2A)*

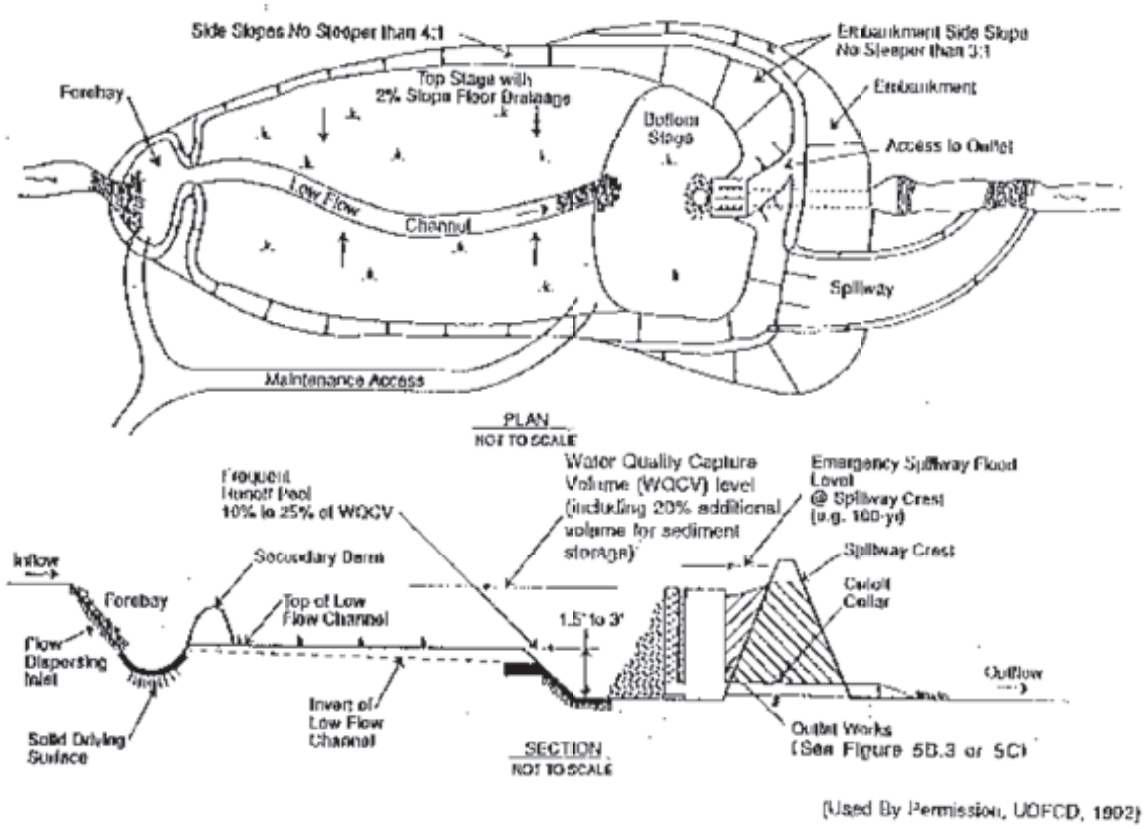
**b - The proposed EDB shall be designed to have a drawdown time or 48 hours, as described in the Geosyntec Report*

A comparison of the required treatment volumes and the volume available within the detention basin clearly indicates that this approach is feasible. The detailed design of the extended bio basin, bio basin areas, and biofilter swales for the project is beyond the scope of this report. However, this report does show that the 80% capture requirements can be met within the areas allocated for the proposed BMPs.

For contingency design, additional approved BMPs (pervious concrete, catch basin insert, etc) approved may be added to subarea RVW-28D (commercial lot) if ED size on BMP8 is reduced during the final engineering process. The same approach applied to multifamily lot on subarea RVC-1A and RVC-9B if BMP 6 is reduced during the final engineering process. However, analysis of this approach is beyond the scope of this preliminary analysis. It should be noted that the SUSMP requirements were calculated for all areas of the project site.

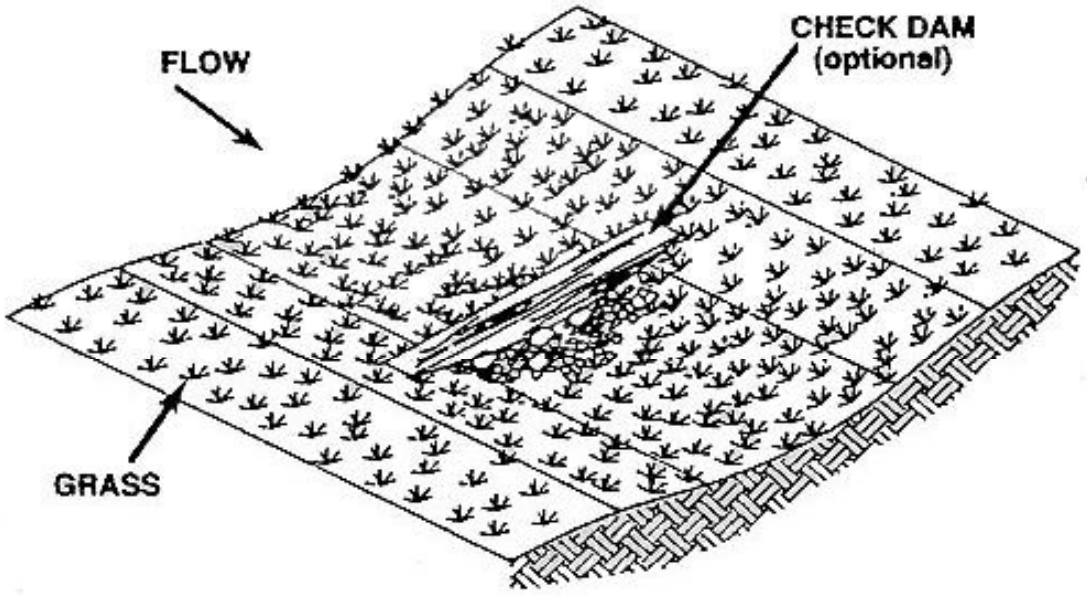
Both bio basin areas and vegetative swales will require flow spreaders so that stormwater flows from the inlet pipes will extend across the width of the swale or bio basin area and reduce the chance of channeling caused by concentrated flows.

Typical layouts of the proposed BMPs are shown below based on LA County SUSMP Manual. Project specific Typical Bio Basin and Extended Bio Basin cross sections are shown on SUSMP Map (Sheet 2A) on Appendix 8.

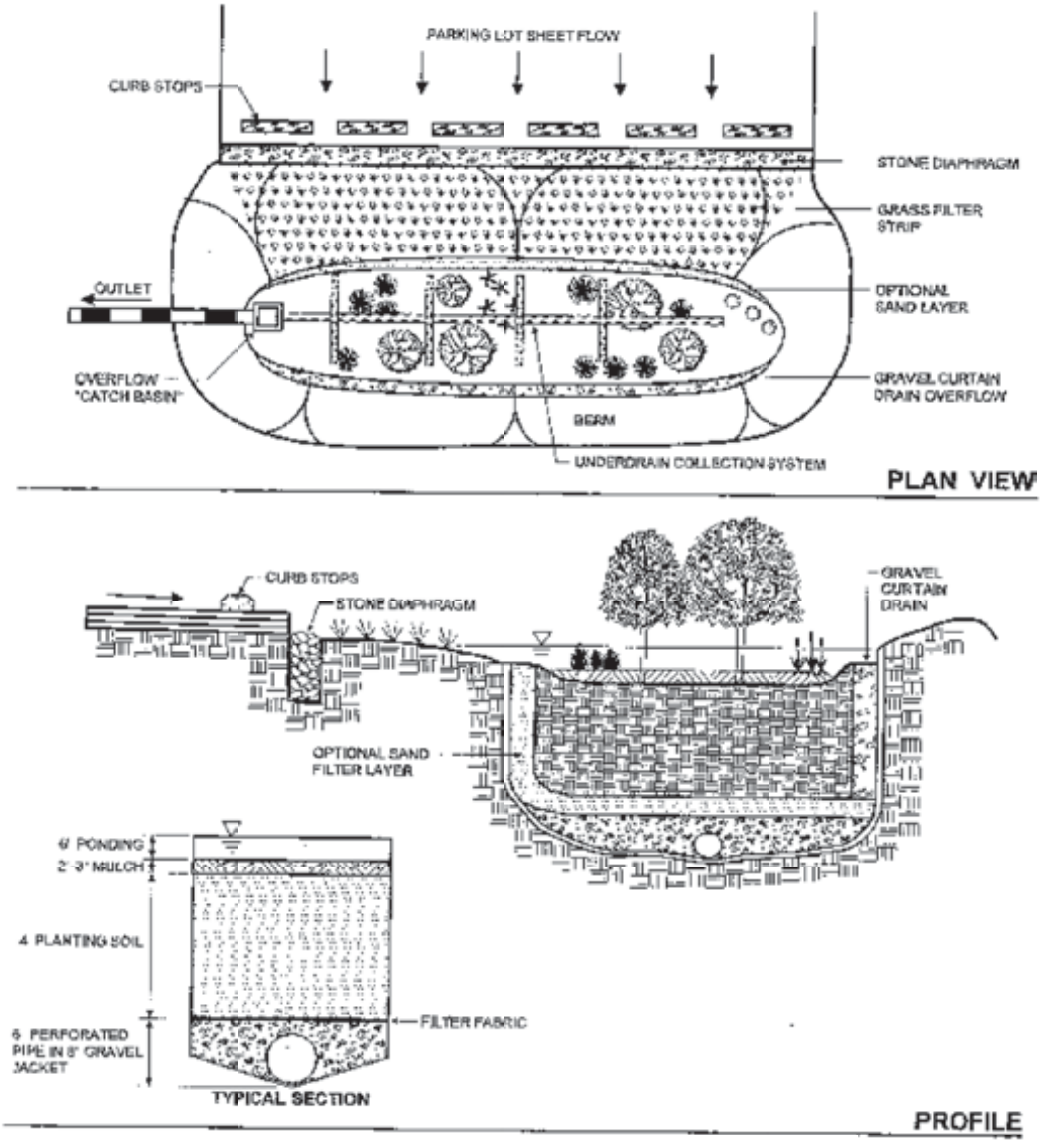


(Used By Permission, UDFCD, 1992)

PLAN AND SECTION OF AN EXTENDED DETENTION BASIN



TYPICAL VEGETATED SWALE



Schematic of a Bioretention Facility (MDE, 2000)

5.0 Limitations

This report was prepared to comply with the guidelines established by the County of Los Angeles and their representatives. Evaluation of the appropriateness of these guidelines and the accuracy of the County data were beyond the scope of this work.

Usage of the report is limited to address the purpose and scope previously defined by the project owner, Newhall Land. Psomas shall not be held responsible for any unauthorized application of this report and the contents herein.

The opinions represented in this report have been derived in accordance with current standards of civil engineering practice. No other warranty is expressed or implied.

6.0 References

California Stormwater Quality Association, *Stormwater Best Management Practice Handbook, New Development and Redevelopment*, January 2003

Los Angeles County Department of Public Works, *LACDPW Hydrology/ Sedimentation Manual and Appendices* (1991, 1992, 1993, 2002)

Los Angeles County Department of Public Works, *LACDPW TC v1.0 Manual, TC_calc_depth.xls* (December 1991, June 2002)

Los Angeles County Department of Public Works, *LACDPW Modified Rational Method, MORA (MORA) v7.2 Manual* (September 2002)

Los Angeles Regional Water Quality Control Board, *Standard Urban Stormwater Mitigation Plan for Los Angeles County and Cities in Los Angeles County* (March 2000)

Newhall Land and Farming Company, Landmark Village Water Quality Technical Report (GeoSyntec Consultants, April 10, 2005)

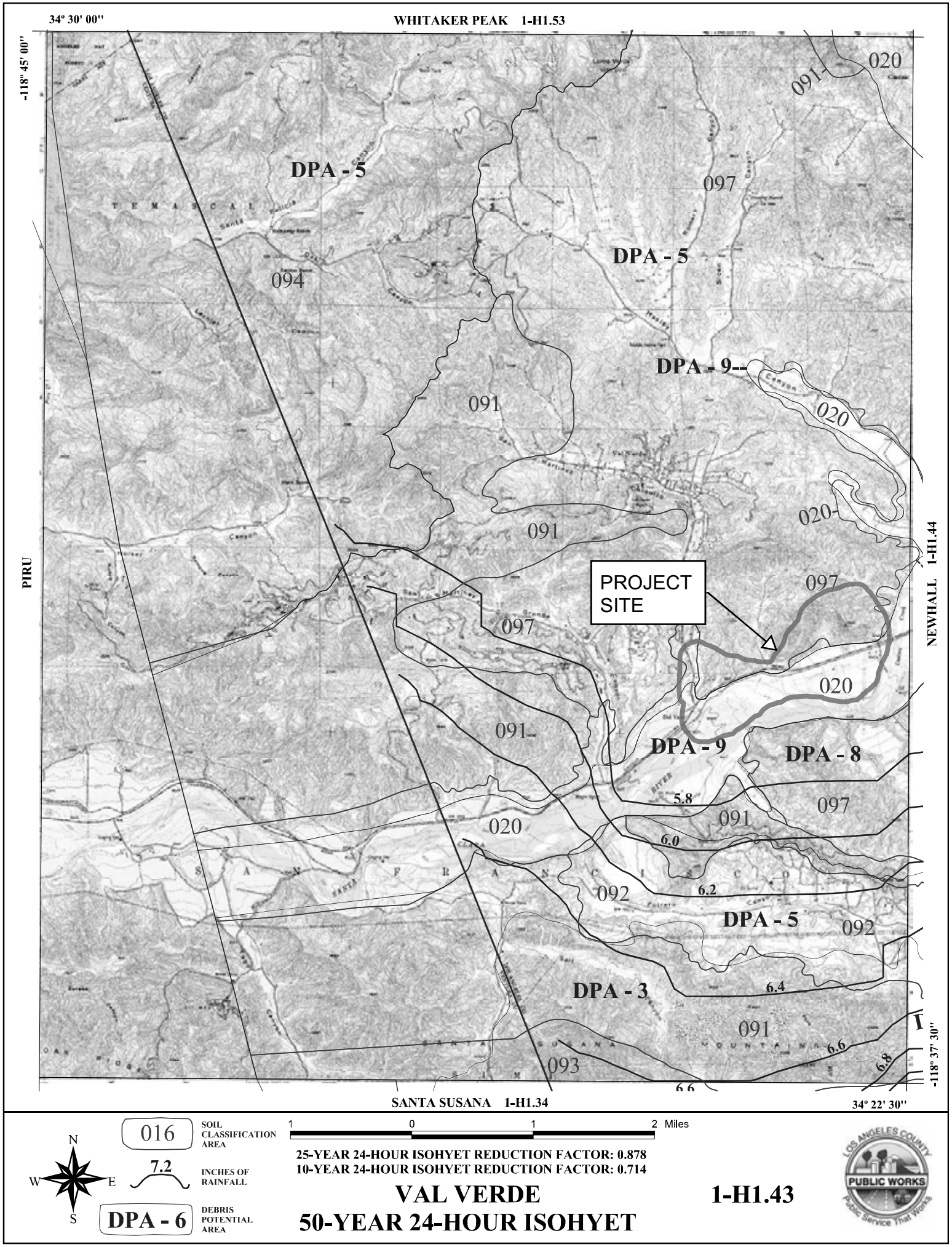
Newhall Ranch, Newhall Ranch Specific Plan Master Hydrology and Drainage Concept (Sikand, December, 1999)

County of Los Angeles Low-Impact Development Manual

APPENDICES

APPENDIX 1

LACDPW HYDROLOGY DATA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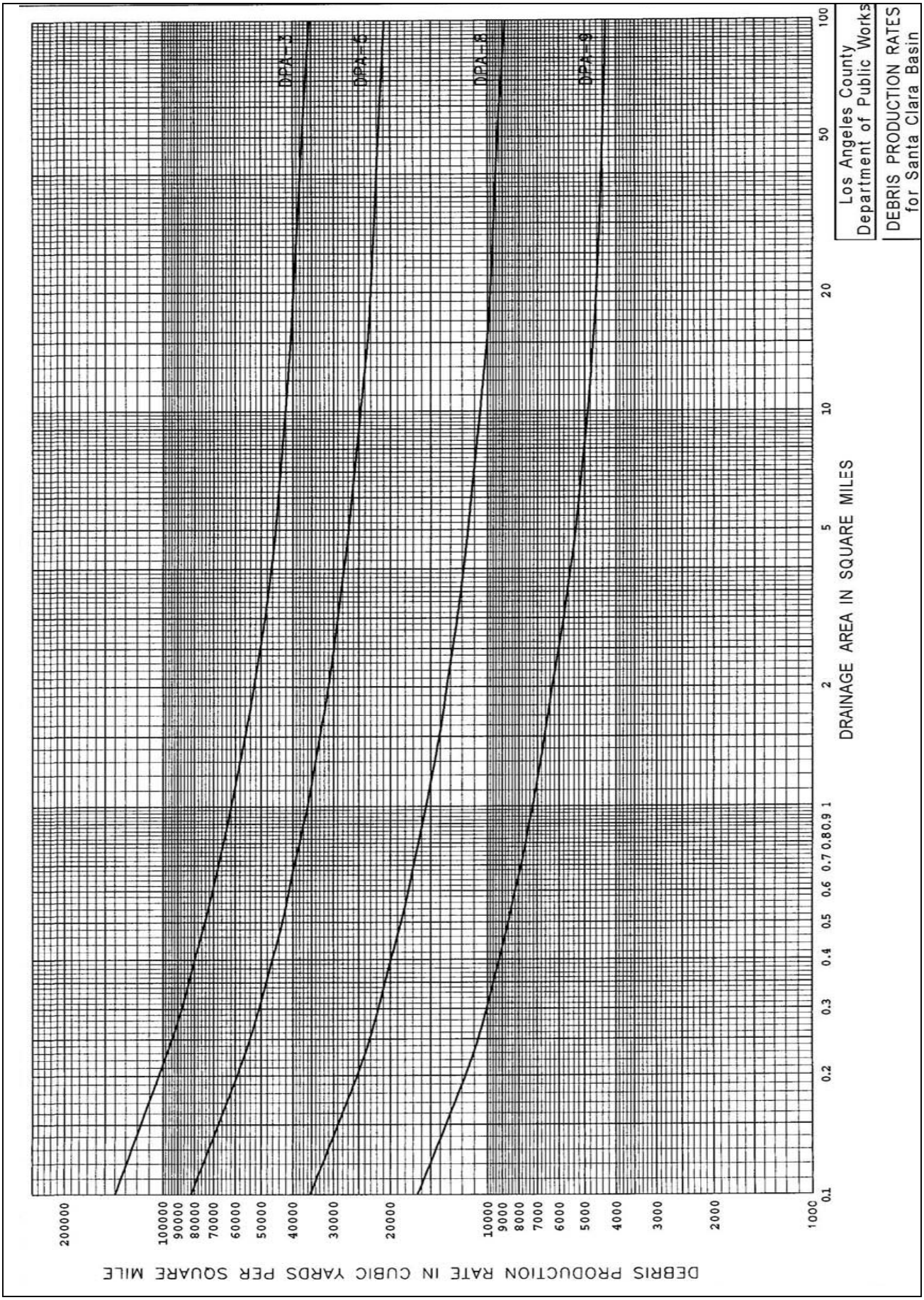
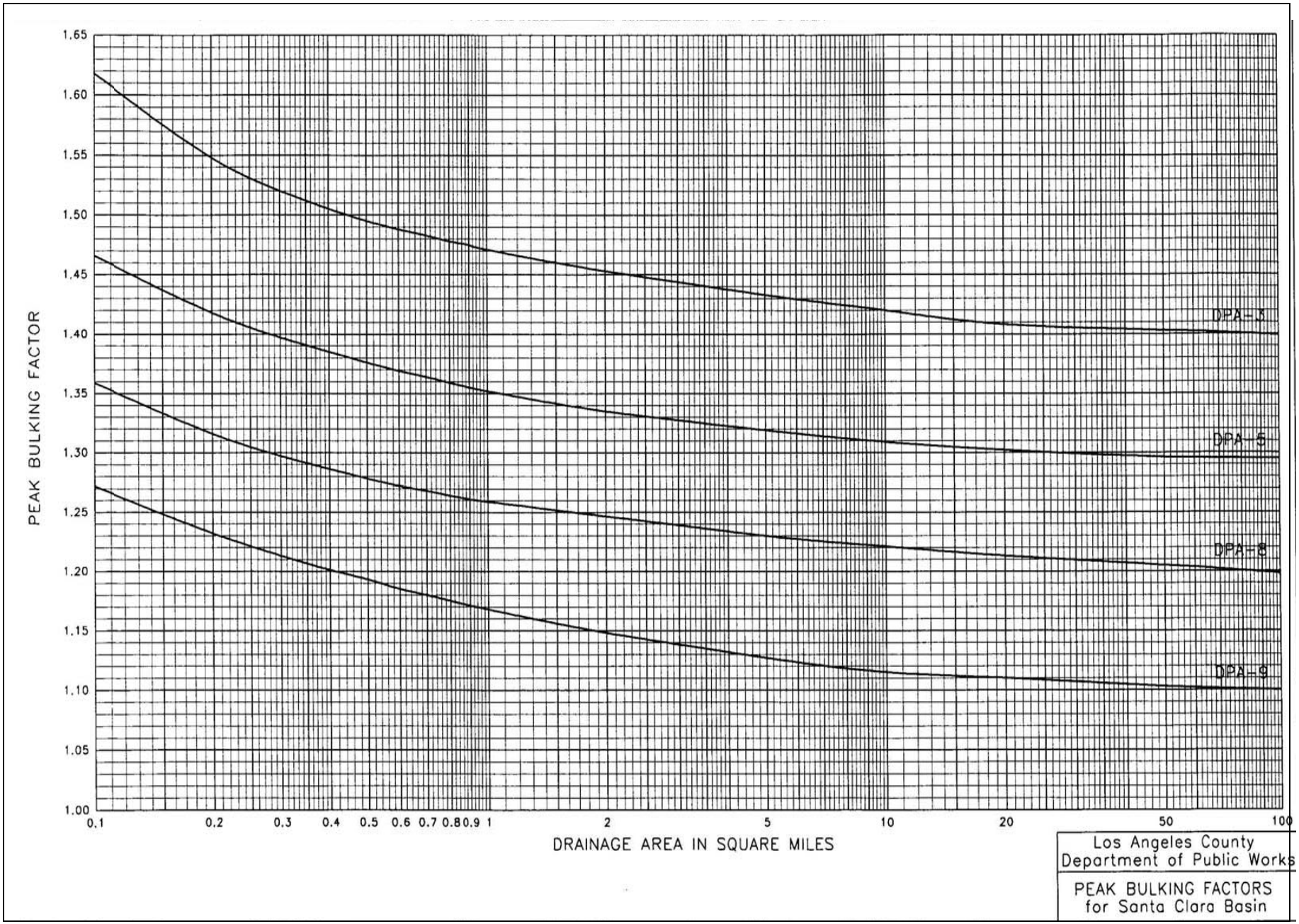
-118° 45' 00"
PIRU
NEWHALL 1-H1.44
-118° 37' 30"
SANTA SUSANA 1-H1.34
34° 22' 30"
DPA - 5
DPA - 5
DPA - 9
DPA - 9
DPA - 8
DPA - 5
DPA - 3
PROJECT SITE
091
020
097
094
092
093
5.8
6.0
6.2
6.4
6.6
6.8
6.6
SANTA SUSANA MOUNTAINS
SAN FERNANDO
016
SOIL CLASSIFICATION AREA
7.2
INCHES OF RAINFALL
DPA - 6
DEBRIS POTENTIAL AREA
N
W
E
S
1 0 1 2 Miles
25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878
10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714
VAL VERDE
50-YEAR 24-HOUR ISOHYET
1-H1.43
LOS ANGELES COUNTY
PUBLIC WORKS
Public Service That Works


DEBRIS PRODUCTION RATE IN CUBIC YARDS PER SQUARE MILE
200000
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
DPA-1
DPA-2
DPA-3
DPA-4
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
5
10
20
50
100
DRAINAGE AREA IN SQUARE MILES
Los Angeles County
Department of Public Works
DEBRIS PRODUCTION RATES
for Santa Clara Basin


PEAK BULKING FACTOR
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1.60
1.55
1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
5
10
20
50
100
DRAINAGE AREA IN SQUARE MILES
DPA-3
DPA-5
DPA-8
DPA-9
Los Angeles County
Department of Public Works
PEAK BULKING FACTORS
for Santa Clara Basin

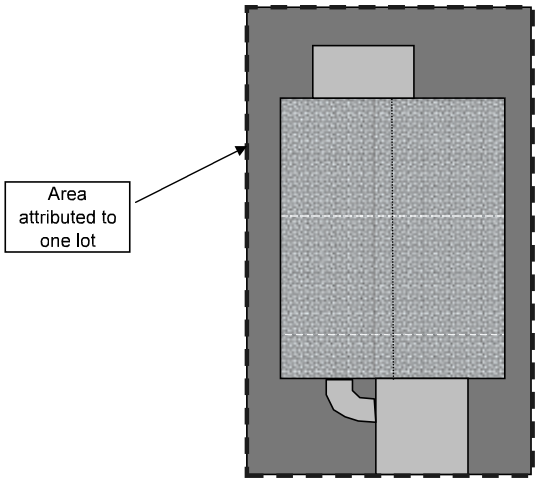
Land Use Impervious Factors

Land Use	Land Use Grouping for WQ and Hydro	Land Use %Imp	Source	Notes
40’ x 95’ SFD	High Density Single Family	58	Study of typical lot dimensions	
45' x 100' SFD				
50' x 100' SFD				
55' x 100' SFD				Incorporates minor roadways
Duplex/Triplex	Multi-family	74	Hydrology Manual listing for Mixed Multi-Family; confirmed based on detailed study of proposed site designs	
Apartments/Affordable Apartments/Condos/Attached Condos				
School	School	82	Hydrology Manual	Incorporates minor roadways
Mixed Commercial	Commercial	91	Hydrology Manual	
Future Metrolink Lot	Roads and Parking Lots	91	Hydrology Manual	Minor roads will be lumped with adjacent residential land uses; 91% accounts for medians and sidewalk buffers in larger ROWs
Future Public Drive			Hydrology Manual	
Private Driveway and Fire Lane			Hydrology Manual	
Trail Head			Hydrology Manual (assumed paved parking lot)	
Park and Ride Lot			Hydrology Manual	
Fire Station	Public Facility	91	Hydrology Manual	
Public Facility			Hydrology Manual	
Recreation	Recreation	50	Based on review of site designs	
Park	Neighborhood Park	15	Hydrology Manual	
Open Space	Open Space	1 or 0	Hydrology Manual	
Development Buffer Zone	Open Space		Hydrology Manual	

Calculations of typical imperviousness in residential land uses
Modify inputs on site-specific basis through inspection of VTTM drawings

Average Lot Area, sf	Avg Lot Dimensions		Set backs			Driveway Area	Lot Area	Lot Imp Area	**Total %Imp**
	Width (ft)	Depth (ft)	Sides (ft)	Front (ft)	Back (ft)	(sf)	(sf)	(sf)	
4500	45	100	5	18	15	288	4500	2633	**58.5%**
5000	50	100	5	18	18	288	5000	2848	**57.0%**
5500	55	100	5	18	18	288	5500	3168	**57.6%**

58%

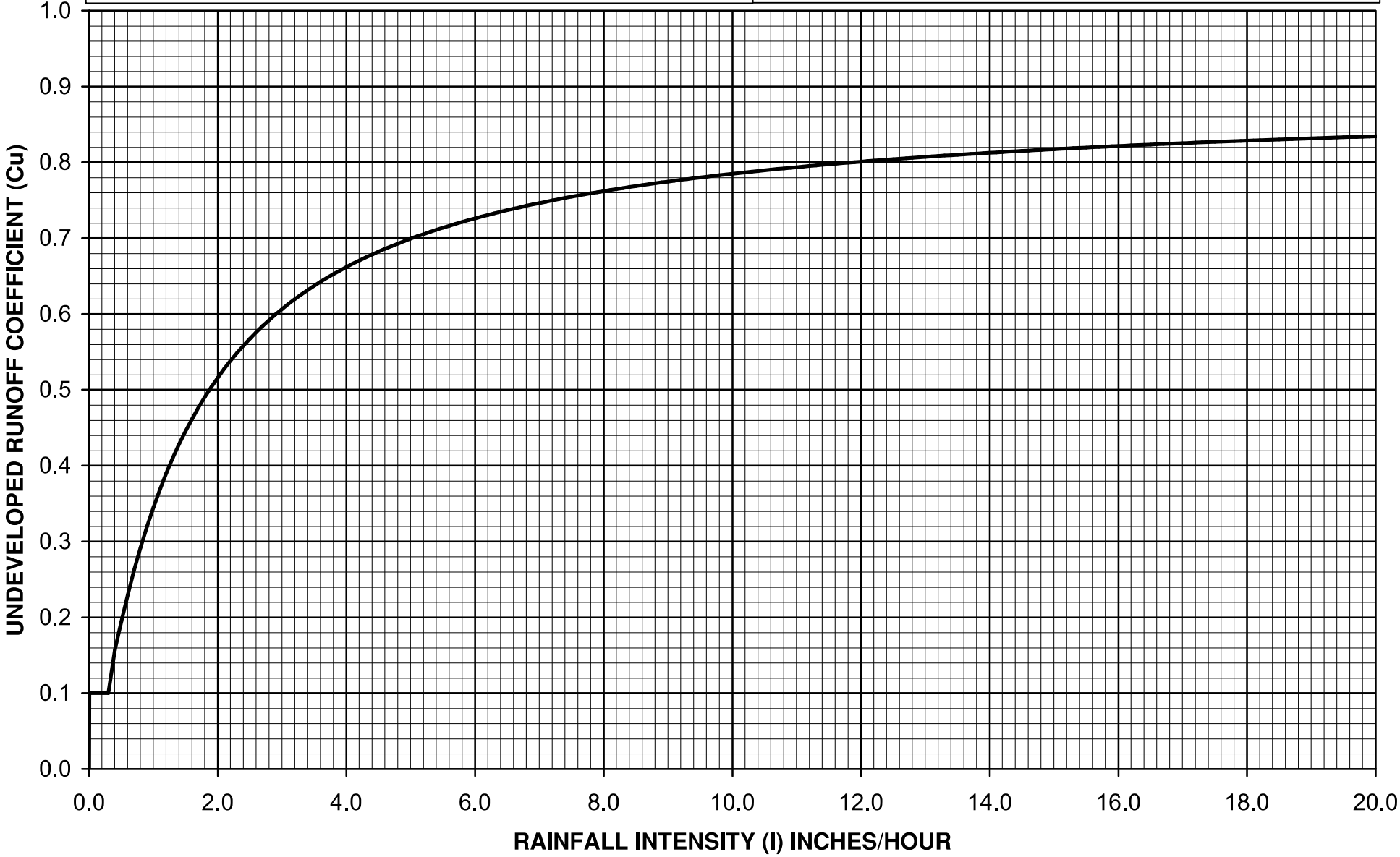


$C_D = (0.9 * IMP) + (1.0 - IMP) * C_U$

Where: C_D = Developed Runoff Coefficient
IMP = Proportion Impervious
C_U = Undeveloped runoff coefficient

Los Angeles County Department of Public Works

RUNOFF COEFFICIENT CURVE
SOIL TYPE NO. 020

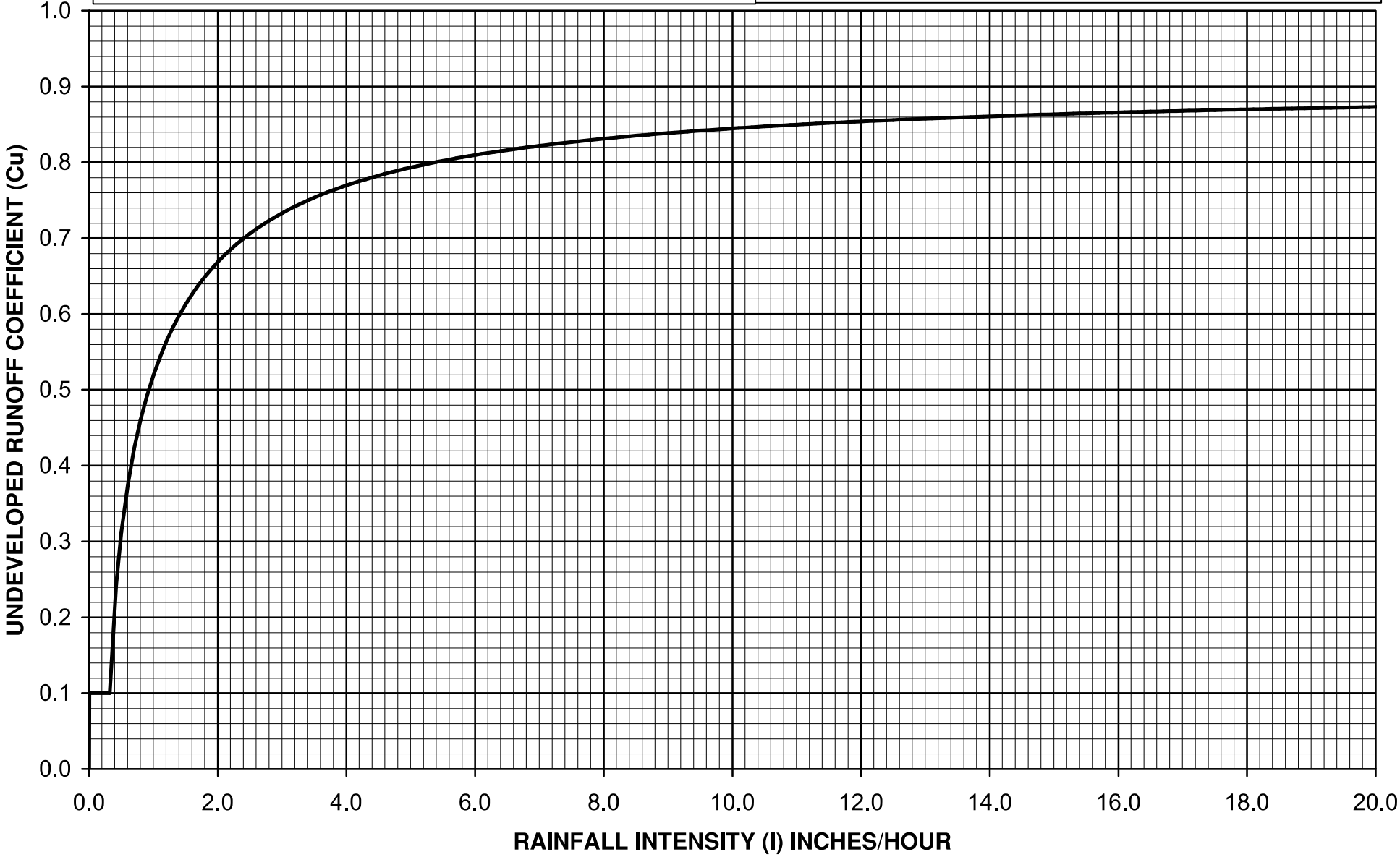


$C_D = (0.9 * IMP) + (1.0 - IMP) * C_U$

Where: C_D = Developed Runoff Coefficient
IMP = Proportion Impervious
C_U = Undeveloped runoff coefficient

Los Angeles County Department of Public Works

RUNOFF COEFFICIENT CURVE
SOIL TYPE NO. 097



APPENDIX 2

TC CALCULATIONS:
EXISTING CONDITION

Landmark Village TR53108
Existing Condition
TC Data File

Project	Subarea	Area	%imp	Frequency	Soil Type	Length	Slope	Isohyet
Area (E)	100A	32.7	0	50	20	1350	0.007	5.8
Area (E)	110A	49.6	0	50	20	1250	0.008	5.8
Area (E)	200A	17.3	0	50	20	1250	0.016	5.8
Area (E)	210A	35.8	0	50	20	1990	0.015	5.8
Area (E)	400A	18.4	0	50	20	2410	0.034	5.8
Area (E)	405A	38.9	0	50	20	3600	0.068	5.8
Area (E)	408C	15.3	0	50	20	1050	0.410	5.8
Area (E)	410C	44.3	0	50	20	1990	0.048	5.8
Area (E)	415B	35.3	0	50	20	1360	0.200	5.8
Area (E)	420A	34.4	0	50	20	1950	0.015	5.8
Area (E)	425A	39.9	0	50	20	1900	0.030	5.8
Area (E)	500A	26.5	0	50	20	1560	0.016	5.8
Area (E)	510A	40	0	50	20	1540	0.006	5.8
Area (E)	CQT-1A	6.1	0	50	20	890	0.265	5.8
Area (E)	CQT-2A	3.6	0	50	20	720	0.317	5.8
Area (E)	CQT-3A	1.8	0	50	20	530	0.336	5.8
Area (E)	CQT-4A	12.3	0	50	20	1160	0.229	5.8
Area (E)	CQT-5A	4.4	0	50	20	530	0.321	5.8
Area (E)	CQT-6A	24.9	0	50	20	1620	0.069	5.8
Area (E)	CQT-7A	2.1	0	50	20	490	0.269	5.8
Area (E)	CQT-8A	2.8	0	50	20	480	0.208	5.8
Area (E)	CQT-9A	31.8	0	50	20	1750	0.150	5.8
Area (E)	CQT-10A	15.6	0	50	20	1300	0.155	5.8
Area (E)	CQT-11A	10.2	65	50	20	2370	0.018	5.8
Area (E)	CQT-12A	11.7	65	50	20	1250	0.032	5.8
Area (E)	620A	12.4	0	50	20	2120	0.030	5.8

Landmark Village TR53108
Existing Condition
TC Calculation Results

Project	Subarea	Area (acre	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft)	Isohyet (in.	Tc-calculat	Intensity (ir	Cu	Cd	Flowrate (cfs)	Tc Equation		
Area (E)	100A	32.7	0	50	20	1350	0.007	5.8	22	1.72	0.48	0.48	27	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	110A	49.6	0	50	20	1250	0.008	5.8	20	1.8	0.49	0.49	43.75	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	200A	17.3	0	50	20	1250	0.016	5.8	17	1.95	0.51	0.51	17.2	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	210A	35.8	0	50	20	1990	0.015	5.8	24	1.66	0.47	0.47	27.93	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	400A	18.4	0	50	20	2410	0.034	5.8	24	1.66	0.47	0.47	14.36	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	405A	38.9	0	50	20	3600	0.068	5.8	28	1.54	0.45	0.45	26.96	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	408C	15.3	0	50	20	1050	0.41	5.8	8	2.77	0.59	0.59	25	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	410C	44.3	0	50	20	1990	0.048	5.8	19	1.85	0.5	0.5	40.98	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	415B	35.3	0	50	20	1360	0.2	5.8	11	2.39	0.55	0.55	46.4	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	420A	34.4	0	50	20	1950	0.015	5.8	24	1.66	0.47	0.47	26.84	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	425A	39.9	0	50	20	1900	0.03	5.8	20	1.8	0.49	0.49	35.19	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	500A	26.5	0	50	20	1560	0.016	5.8	20	1.8	0.49	0.49	23.37	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	510A	40	0	50	20	1540	0.006	5.8	24	1.66	0.47	0.47	31.21	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-1A	6.1	0	50	20	890	0.265	5.8	8	2.77	0.59	0.59	9.97	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-2A	3.6	0	50	20	720	0.317	5.8	6	3.18	0.62	0.62	7.1	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-3A	1.8	0	50	20	530	0.336	5.8	5	3.46	0.63	0.63	3.92	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-4A	12.3	0	50	20	1160	0.229	5.8	10	2.5	0.56	0.56	17.22	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-5A	4.4	0	50	20	530	0.321	5.8	5	3.46	0.63	0.63	9.59	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-6A	24.9	0	50	20	1620	0.069	5.8	15	2.06	0.52	0.52	26.67	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-7A	2.1	0	50	20	490	0.269	5.8	5	3.46	0.63	0.63	4.58	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-8A	2.8	0	50	20	480	0.208	5.8	5	3.46	0.63	0.63	6.1	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-9A	31.8	0	50	20	1750	0.15	5.8	14	2.13	0.53	0.53	35.9	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-10A	15.6	0	50	20	1300	0.155	5.8	11	2.39	0.55	0.55	20.51	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-11A	10.2	65	50	20	2370	0.018	5.8	17	1.95	0.51	0.9	17.9	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	CQT-12A	11.7	65	50	20	1250	0.032	5.8	10	2.5	0.56	0.9	26.33	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		
Area (E)	620A	12.4	0	50	20	2120	0.03	5.8	22	1.72	0.48	0.48	10.24	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^		

APPENDIX 3

TC CALCULATIONS:
PROPOSED CONDITION

Landmark Village TR53108
Proposed Condition
TC Data File

Project	Subarea	Area (Acres)	% Impervious	Storm Frequency	Soil Type	Length (Ft)	Slope	Isohyet
RVE	1A	18.40	0.00	50	97	2,430	0.0340	5.8
RVE	2A	38.90	0.00	50	97	3,550	0.0690	5.8
RVE	3B	15.30	0.00	50	97	1,050	0.4100	5.8
RVE	4B	44.30	0.00	50	97	1,990	0.0500	5.8
RVE	6A	35.30	0.00	50	97	1,360	0.2000	5.8
RVE	7A	15.53	1.00	50	20	4,200	0.0070	5.8
RVE	8A	6.35	15.00	50	20	656	0.0106	5.8
RVE	9B	7.04	65.64	50	20	761	0.0070	5.8
RVE	10B	0.69	72.74	50	20	333	0.0070	5.8
RVE	11C	1.02	63.18	50	20	238	0.0170	5.8
RVE	12C	0.63	65.86	50	20	238	0.0170	5.8
RVE	13C	0.87	65.97	50	20	303	0.0070	5.8
RVE	14C	0.20	91.00	50	20	277	0.0070	5.8
RVE	15C	0.54	91.00	50	20	760	0.0080	5.8
RVE	16C	2.78	64.17	50	20	771	0.0080	5.8
RVE	17C	5.17	68.75	50	20	656	0.0110	5.8
RVE	18C	0.66	91.00	50	20	969	0.0090	5.8
RVE	19C	0.30	91.00	50	20	432	0.0090	5.8
RVE	20C	4.68	16.00	50	20	1,749	0.0050	5.8
RVE	21D	1.82	64.16	50	20	495	0.0080	5.8
RVE	22D	0.34	91.00	50	20	476	0.0080	5.8
RVE	22.1D	3.56	74.00	50	20	538	0.0100	5.8
RVE	23D	0.55	91.00	50	20	794	0.0080	5.8
RVE	24D	0.77	91.00	50	20	1,099	0.0090	5.8
RVE	25D	0.21	91.00	50	20	214	0.0080	5.8
RVE	26D	5.09	74.00	50	20	1,036	0.0110	5.8
RVE	27D	0.95	91.00	50	20	1,309	0.0090	5.8
RVE	28D	1.78	72.83	50	20	867	0.0070	5.8
RVE	29D	2.08	14.41	50	20	832	0.0050	5.8
RVE	30E	0.53	91.00	50	20	689	0.0090	5.8
RVE	31E	0.47	91.00	50	20	236	0.0080	5.8
RVE	32E	0.33	91.00	50	20	109	0.0060	5.8
RVE	33E	5.86	70.70	50	20	785	0.0110	5.8
RVE	34E	5.29	71.29	50	20	557	0.0110	5.8
RVE	35E	1.23	70.34	50	20	626	0.0080	5.8
RVE	36E	0.68	91.00	50	20	808	0.0090	5.8
RVE	37E	1.70	16.88	50	20	652	0.0050	5.8
RVE	38F	0.25	91.00	50	20	361	0.0090	5.8
RVE	39F	2.03	74.00	50	20	399	0.0120	5.8
RVE	40F	0.05	91.00	50	20	71	0.0070	5.8
RVE	41F	1.79	67.03	50	20	709	0.0080	5.8
RVE	42F	0.50	91.00	50	20	663	0.0070	5.8
RVE	43F	3.86	67.66	50	20	1,117	0.0080	5.8
RVE	44F	2.86	66.42	50	20	915	0.0070	5.8
RVE	45F	0.14	91.00	50	20	118	0.0070	5.8
RVE	46F	2.69	17.39	50	20	1,120	0.0050	5.8
RVE	51G	2.99	67.27	50	20	570	0.0080	5.8
RVE	52G	1.24	66.31	50	20	460	0.0070	5.8
RVE	53G	1.21	65.36	50	20	340	0.0070	5.8
RVE	54G	1.19	65.49	50	20	344	0.0070	5.8

Landmark Village TR53108
Proposed Condition
TC Data File

Project	Subarea	Area (Acres)	% Impervious	Storm Frequency	Soil Type	Length (Ft)	Slope	Isohyet
RVE	1A	18.40	0.00	50	97	2,430	0.0340	5.8
RVE	55G	0.82	64.44	50	20	235	0.0070	5.8
RVE	56G	1.02	69.00	50	20	485	0.0090	5.8
RVE	57G	3.37	67.79	50	20	666	0.0070	5.8
RVE	58G	1.93	73.28	50	20	1,218	0.0100	5.8
RVE	59G	1.90	52.16	50	20	781	0.0080	5.8
RVE	60G	9.88	74.00	50	20	961	0.0090	5.8
RVE	61G	1.36	91.00	50	20	695	0.0100	5.8
RVE	62G	1.22	91.00	50	20	580	0.0100	5.8
RVE	63G	4.11	70.85	50	20	802	0.0080	5.8
RVC	1A	11.66	78.91	50	20	1,103	0.0070	5.8
RVC	2A	4.08	74.00	50	20	697	0.0100	5.8
RVC	3A	2.39	91.00	50	20	1,547	0.0110	5.8
RVC	4A	1.67	91.00	50	20	1,260	0.0100	5.8
RVC	5A	0.40	91.00	50	20	213	0.0070	5.8
RVC	6A	0.78	91.00	50	20	335	0.0100	5.8
RVC	7A	0.55	50.00	50	20	100	0.0080	5.8
RVC	8A	1.52	15.80	50	20	400	0.0050	5.8
RVC	9B	8.39	74.00	50	20	1,014	0.0100	5.8
RVC	10B	1.03	91.00	50	20	650	0.0070	5.8
RVC	11B	0.74	91.00	50	20	602	0.0070	5.8
RVC	12B	9.79	82.00	50	20	850	0.0080	5.8
RVC	13B	1.34	91.00	50	20	840	0.0070	5.8
RVC	14B	2.83	71.01	50	20	880	0.0070	5.8
RVC	15C	3.54	15.00	50	20	560	0.0080	5.8
RVW	1A	0.82	91.00	50	20	383	0.0070	5.8
RVW	2A	3.21	74.00	50	20	335	0.0100	5.8
RVW	3A	1.48	69.15	50	20	485	0.0070	5.8
RVW	4A	3.34	63.73	50	20	550	0.0070	5.8
RVW	5B	0.34	91.00	50	20	267	0.0070	5.8
RVW	6B	1.21	74.00	50	20	250	0.0070	5.8
RVW	7B	0.18	91.00	50	20	221	0.0180	5.8
RVW	8B	0.20	91.00	50	20	225	0.0180	5.8
RVW	9B	2.22	70.78	50	20	481	0.0070	5.8
RVW	10B	1.33	76.54	50	20	462	0.0070	5.8
RVW	11B	0.34	65.76	50	20	85	0.0080	5.8
RVW	12B	3.54	16.00	50	20	1,360	0.0060	5.8
RVW	12C	0.87	91.00	50	20	1,000	0.0070	5.8
RVW	13C	3.60	65.43	50	20	597	0.0070	5.8
RVW	14C	2.58	68.23	50	20	1,147	0.0080	5.8
RVW	15C	2.13	65.28	50	20	650	0.0090	5.8
RVW	16C	0.99	91.00	50	20	466	0.0070	5.8
RVW	17C	1.91	15.61	50	20	800	0.0050	5.8
RVW	18D	1.93	74.00	50	20	517	0.0110	5.8
RVW	19D	0.51	91.00	50	20	305	0.0070	5.8
RVW	20D	0.38	91.00	50	20	310	0.0070	5.8
RVW	21D	11.00	74.00	50	20	1,570	0.0090	5.8
RVW	22D	1.88	91.00	50	20	1,200	0.0080	5.8
RVW	23D	1.26	91.00	50	20	1,071	0.0090	5.8
RVW	24D	4.91	74.00	50	20	1,000	0.0100	5.8

Landmark Village TR53108
Proposed Condition
TC Data File

Project	Subarea	Area (Acres)	% Impervious	Storm Frequency	Soil Type	Length (Ft)	Slope	Isohyet
RVE	1A	18.40	0.00	50	97	2,430	0.0340	5.8
RVW	25D	0.49	91.00	50	20	320	0.0070	5.8
RVW	26D	1.11	91.00	50	20	220	0.0100	5.8
RVW	26.1D	0.56	91.00	50	20	280	0.0070	5.8
RVW	27D	1.02	91.00	50	20	340	0.0070	5.8
RVW	28D	17.71	74.00	50	20	982	0.0100	5.8
RVW	29D	4.06	74.00	50	20	637	0.0100	5.8
RVW	31E	2.17	91.00	50	20	720	0.0070	5.8
RVW	32E	0.70	91.00	50	20	385	0.0190	5.8
RVW	33E	1.99	91.00	50	20	615	0.0190	5.8
RVW	34E	2.44	91.00	50	20	880	0.0100	5.8
RVW	35E	0.65	91.00	50	20	510	0.0080	5.8
RVW	36E	0.26	91.00	50	20	180	0.0100	5.8
RVW	37E	0.37	91.00	50	20	320	0.0110	5.8
RVW	38E	0.77	91.00	50	20	460	0.0130	5.8
RVW	39E	5.72	78.52	50	20	930	0.0100	5.8
RVW	39.1E	0.78	91.00	50	20	550	0.0100	5.8
RVW	40E	7.41	91.00	50	20	1,050	0.0190	5.8
RVW	41E	2.32	13.03	50	20	730	0.0050	5.8
RVW	42E	1.04	13.12	50	20	400	0.0050	5.8
RVW	43G	1.29	14.95	50	20	540	0.0050	5.8
RVW	44G	1.94	9.81	50	20	140	0.0500	5.8
CQT	1A	29.90	0.05	50	20	1,150	0.0895	5.8
CQT	2A	6.40	0.05	50	20	630	0.2700	5.8
CQT	3B	4.40	0.05	50	20	540	0.2800	5.8
CQT	4B	19.60	0.10	50	20	1,404	0.0560	5.8
CQT	5B	6.94	0.05	50	20	765	0.2100	5.8
CQT	6B	5.90	0.05	50	20	580	0.0600	5.8
CQT	7B	13.30	0.30	50	20	860	0.0600	5.8
CQT	9C	7.70	0.05	50	20	1,050	0.2800	5.8
CQT	10C	8.80	0.05	50	20	970	0.0600	5.8
CQT	11C	11.80	0.80	50	20	1,890	0.0080	5.8
CQT	12C	2.20	0.05	50	20	1,310	0.0240	5.8
CQT	13C	7.50	0.80	50	20	2,500	0.0070	5.8

Landmark Village

Proposed Condition TC-Calc Output File

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft)	Isohyet (in.)	Tc-calculated (min.)	Intensity (in./hr)	Cu	Cd	Flowrate (c	Tc Equation
RVE	1A	18.4	0.0	50	97	2430	0.034	5.8	19	1.85	0.65	0.65	22.13	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	2A	38.9	0.0	50	97	3550	0.069	5.8	22	1.72	0.64	0.64	42.82	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	3B	15.3	0.0	50	97	1050	0.41	5.8	7	2.95	0.73	0.73	32.95	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	4B	44.3	0.0	50	97	1990	0.05	5.8	15	2.06	0.67	0.67	61.14	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	6A	35.3	0.0	50	97	1360	0.2	5.8	9	2.63	0.71	0.71	65.92	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	7A	15.53	1.0	50	20	4200	0.007	5.8	29	1.51	0.45	0.9	21.11	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	8A	6.35	15.0	50	20	656	0.0106	5.8	8	2.77	0.59	0.9	15.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	9B	7.04	65.6	50	20	761	0.007	5.8	10	2.5	0.56	0.9	15.84	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	10B	0.69	72.7	50	20	333	0.007	5.8	6	3.18	0.62	0.9	1.97	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	11C	1.02	63.2	50	20	238	0.017	5.8	5	3.46	0.63	0.9	3.18	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	12C	0.63	65.9	50	20	238	0.017	5.8	5	3.46	0.63	0.9	1.96	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	13C	0.87	66.0	50	20	303	0.007	5.8	5	3.46	0.63	0.9	2.71	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	14C	0.2	91.0	50	20	277	0.007	5.8	5	3.46	0.63	0.9	0.62	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	15C	0.54	91.0	50	20	760	0.008	5.8	10	2.5	0.56	0.9	1.22	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	16C	2.78	64.2	50	20	771	0.008	5.8	10	2.5	0.56	0.9	6.26	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	17C	5.17	68.8	50	20	656	0.011	5.8	8	2.77	0.59	0.9	12.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	18C	0.66	91.0	50	20	969	0.009	5.8	11	2.39	0.55	0.9	1.42	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	19C	0.3	91.0	50	20	432	0.009	5.8	7	2.95	0.6	0.9	0.8	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	20C	4.68	16.0	50	20	1749	0.005	5.8	18	1.9	0.5	0.9	8	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	21D	1.82	64.2	50	20	495	0.008	5.8	7	2.95	0.6	0.9	4.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	22D	0.34	91.0	50	20	476	0.008	5.8	7	2.95	0.6	0.9	0.9	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	22.1D	3.56	74.0	50	20	538	0.01	5.8	7	2.95	0.6	0.9	9.45	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	23D	0.55	91.0	50	20	794	0.008	5.8	10	2.5	0.56	0.9	1.24	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	24D	0.77	91.0	50	20	1099	0.009	5.8	12	2.29	0.54	0.9	1.59	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	25D	0.21	91.0	50	20	214	0.008	5.8	5	3.46	0.63	0.9	0.65	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	26D	5.09	74.0	50	20	1036	0.011	5.8	11	2.39	0.55	0.9	10.95	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	27D	0.95	91.0	50	20	1309	0.009	5.8	13	2.21	0.54	0.9	1.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	28D	1.78	72.8	50	20	867	0.007	5.8	11	2.39	0.55	0.9	3.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	29D	2.08	14.4	50	20	832	0.005	5.8	11	2.39	0.55	0.9	4.47	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	30E	0.53	91.0	50	20	689	0.009	5.8	9	2.63	0.57	0.9	1.25	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	31E	0.47	91.0	50	20	236	0.008	5.8	5	3.46	0.63	0.9	1.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	32E	0.33	91.0	50	20	109	0.006	5.8	5	3.46	0.63	0.9	1.03	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	33E	5.86	70.7	50	20	785	0.011	5.8	9	2.63	0.57	0.9	13.87	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	34E	5.29	71.3	50	20	557	0.011	5.8	7	2.95	0.6	0.9	14.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	35E	1.23	70.3	50	20	626	0.008	5.8	8	2.77	0.59	0.9	3.07	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	36E	0.68	91.0	50	20	808	0.009	5.8	10	2.5	0.56	0.9	1.53	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	37E	1.7	16.9	50	20	652	0.005	5.8	10	2.5	0.56	0.9	3.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	38F	0.25	91.0	50	20	361	0.009	5.8	6	3.18	0.62	0.9	0.72	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	39F	2.03	74.0	50	20	399	0.012	5.8	6	3.18	0.62	0.9	5.81	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	40F	0.05	91.0	50	20	71	0.007	5.8	5	3.46	0.63	0.9	0.16	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	41F	1.79	67.0	50	20	709	0.008	5.8	9	2.63	0.57	0.9	4.24	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	42F	0.5	91.0	50	20	663	0.007	5.8	9	2.63	0.57	0.9	1.18	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	43F	3.86	67.7	50	20	1117	0.008	5.8	12	2.29	0.54	0.9	7.96	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	44F	2.86	66.4	50	20	915	0.007	5.8	11	2.39	0.55	0.9	6.15	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	45F	0.14	91.0	50	20	118	0.007	5.8	5	3.46	0.63	0.9	0.44	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	46F	2.69	17.4	50	20	1120	0.005	5.8	13	2.21	0.54	0.9	5.35	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	51G	2.99	67.3	50	20	570	0.008	5.8	8	2.77	0.59	0.9	7.45	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	52G	1.24	66.3	50	20	460	0.007	5.8	7	2.95	0.6	0.9	3.29	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	53G	1.21	65.4	50	20	340	0.007	5.8	6	3.18	0.62	0.9	3.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	54G	1.19	65.5	50	20	344	0.007	5.8	6	3.18	0.62	0.9	3.41	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	55G	0.82	64.4	50	20	235	0.007	5.8	5	3.46	0.63	0.9	2.55	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	56G	1.02	69.0	50	20	485	0.009	5.8	7	2.95	0.6	0.9	2.71	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	57G	3.37	67.8	50	20	666	0.007	5.8	9	2.63	0.57	0.9	7.98	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	58G	1.93	73.3	50	20	1218	0.01	5.8	12	2.29	0.54	0.9	3.98	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	59G	1.9	52.2	50	20	781	0.008	5.8	10	2.5	0.56	0.9	4.28	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	60G	9.88	74.0	50	20	961	0.009	5.8	11	2.39	0.55	0.9	21.25	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	61G	1.36	91.0	50	20	695	0.01	5.8	9	2.63	0.57	0.9	3.22	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	62G	1.22	91.0	50	20	580	0.01	5.8	8	2.77	0.59	0.9	3.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVE	63G	4.11	70.9	50	20	802	0.008	5.8	10	2.5	0.56	0.9	9.25	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	1A	11.66	78.9	50	20	1103	0.007	5.8	12	2.29	0.54	0.9	24.03	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	2A	4.08	74.0	50	20	697	0.01	5.8	9	2.63	0.57	0.9	9.66	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	3A	2.39	91.0	50	20	1547	0.011	5.8	14	2.13	0.53	0.9	4.58	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	4A	1.67	91.0	50	20	1260	0.01	5.8	13	2.21	0.54	0.9	3.32	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	5A	0.4	91.0	50	20	213	0.007	5.8	5	3.46	0.63	0.9	1.25	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	6A	0.78	91.0	50	20	335	0.01	5.8	5	3.46	0.63	0.9	2.43	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	7A	0.55	50.0	50	20	100	0.008	5.8	5	3.46	0.63	0.9	1.71	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Landmark Village

Proposed Condition TC-Calc Output File

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft)	Isohyet (in.)	Tc-calculated (min.)	Intensity (in./hr)	Cu	Cd	Flowrate (c	Tc Equation
RVC	8A	1.52	15.8	50	20	400	0.005	5.8	7	2.95	0.6	0.9	4.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	9B	8.39	74.0	50	20	1014	0.01	5.8	11	2.39	0.55	0.9	18.05	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	10B	1.03	91.0	50	20	650	0.007	5.8	9	2.63	0.57	0.9	2.44	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	11B	0.74	91.0	50	20	602	0.007	5.8	8	2.77	0.59	0.9	1.84	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	12B	9.79	82.0	50	20	850	0.008	5.8	10	2.5	0.56	0.9	22.03	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	13B	1.34	91.0	50	20	840	0.007	5.8	11	2.39	0.55	0.9	2.88	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	14B	2.83	71.0	50	20	880	0.007	5.8	11	2.39	0.55	0.9	6.09	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVC	15C	3.54	15.0	50	20	560	0.008	5.8	8	2.77	0.59	0.9	8.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	1A	0.82	91.0	50	20	383	0.007	5.8	6	3.18	0.62	0.9	2.35	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	2A	3.21	74.0	50	20	335	0.01	5.8	5	3.46	0.63	0.9	10	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	3A	1.48	69.1	50	20	485	0.007	5.8	7	2.95	0.6	0.9	3.93	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	4A	3.34	63.7	50	20	550	0.007	5.8	8	2.77	0.59	0.9	8.33	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	5B	0.34	91.0	50	20	267	0.007	5.8	5	3.46	0.63	0.9	1.06	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	6B	1.21	74.0	50	20	250	0.007	5.8	5	3.46	0.63	0.9	3.77	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	7B	0.18	91.0	50	20	221	0.018	5.8	5	3.46	0.63	0.9	0.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	8B	0.2	91.0	50	20	225	0.018	5.8	5	3.46	0.63	0.9	0.62	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	9B	2.22	70.8	50	20	481	0.007	5.8	7	2.95	0.6	0.9	5.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	10B	1.33	76.5	50	20	462	0.007	5.8	7	2.95	0.6	0.9	3.53	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	11B	0.34	65.8	50	20	85	0.008	5.8	5	3.46	0.63	0.9	1.06	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	12B	3.54	16.0	50	20	1360	0.006	5.8	15	2.06	0.52	0.9	6.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	12C	0.87	91.0	50	20	1000	0.007	5.8	12	2.29	0.54	0.9	1.79	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	13C	3.6	65.4	50	20	597	0.007	5.8	8	2.77	0.59	0.9	8.97	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	14C	2.58	68.2	50	20	1147	0.008	5.8	12	2.29	0.54	0.9	5.32	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	15C	2.13	65.3	50	20	650	0.009	5.8	9	2.63	0.57	0.9	5.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	16C	0.99	91.0	50	20	466	0.007	5.8	7	2.95	0.6	0.9	2.63	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	17C	1.91	15.6	50	20	800	0.005	5.8	11	2.39	0.55	0.9	4.11	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	18D	1.93	74.0	50	20	517	0.011	5.8	7	2.95	0.6	0.9	5.12	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	19D	0.51	91.0	50	20	305	0.007	5.8	6	3.18	0.62	0.9	1.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	20D	0.38	91.0	50	20	310	0.007	5.8	6	3.18	0.62	0.9	1.09	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	21D	11	74.0	50	20	1570	0.009	5.8	15	2.06	0.52	0.9	20.39	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	22D	1.88	91.0	50	20	1200	0.008	5.8	13	2.21	0.54	0.9	3.74	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	23D	1.26	91.0	50	20	1071	0.009	5.8	12	2.29	0.54	0.9	2.6	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	24D	4.91	74.0	50	20	1000	0.01	5.8	11	2.39	0.55	0.9	10.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	25D	0.49	91.0	50	20	320	0.007	5.8	6	3.18	0.62	0.9	1.4	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	26D	1.11	91.0	50	20	220	0.01	5.8	5	3.46	0.63	0.9	3.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	26.1D	0.56	91.0	50	20	280	0.007	5.8	5	3.46	0.63	0.9	1.74	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	27D	1.02	91.0	50	20	340	0.007	5.8	6	3.18	0.62	0.9	2.92	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	28D	17.71	74.0	50	20	982	0.01	5.8	11	2.39	0.55	0.9	38.09	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	29D	4.06	74.0	50	20	637	0.01	5.8	8	2.77	0.59	0.9	10.12	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	31E	2.17	91.0	50	20	720	0.007	5.8	10	2.5	0.56	0.9	4.88	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	32E	0.7	91.0	50	20	385	0.019	5.8	5	3.46	0.63	0.9	2.18	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	33E	1.99	91.0	50	20	615	0.019	5.8	7	2.95	0.6	0.9	5.28	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	34E	2.44	91.0	50	20	880	0.01	5.8	10	2.5	0.56	0.9	5.49	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	35E	0.65	91.0	50	20	510	0.008	5.8	7	2.95	0.6	0.9	1.73	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	36E	0.26	91.0	50	20	180	0.01	5.8	5	3.46	0.63	0.9	0.81	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	37E	0.37	91.0	50	20	320	0.011	5.8	5	3.46	0.63	0.9	1.15	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	38E	0.77	91.0	50	20	460	0.013	5.8	6	3.18	0.62	0.9	2.2	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	39E	5.72	78.5	50	20	930	0.01	5.8	11	2.39	0.55	0.9	12.3	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	39.1E	0.78	91.0	50	20	550	0.01	5.8	8	2.77	0.59	0.9	1.94	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	40E	7.41	91.0	50	20	1050	0.019	5.8	10	2.5	0.56	0.9	16.67	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	41E	2.32	13.0	50	20	730	0.005	5.8	10	2.5	0.56	0.9	5.22	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	42E	1.04	13.1	50	20	400	0.005	5.8	7	2.95	0.6	0.9	2.76	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	43G	1.29	15.0	50	20	540	0.005	5.8	8	2.77	0.59	0.9	3.22	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
RVW	44G	1.94	9.8	50	20	140	0.05	5.8	5	3.46	0.63	0.9	6.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	1A	29.9	0.1	50	20	1150	0.0895	5.8	11	2.39	0.55	0.57	40.73	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	2A	6.4	0.1	50	20	630	0.27	5.8	6	3.18	0.62	0.63	12.82	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	3B	4.4	0.1	50	20	540	0.28	5.8	5	3.46	0.63	0.64	9.74	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	4B	19.6	0.1	50	20	1404	0.056	5.8	14	2.13	0.53	0.57	23.8	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	5B	6.94	0.1	50	20	765	0.21	5.8	7	2.95	0.6	0.62	12.69	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	6B	5.9	0.1	50	20	580	0.06	5.8	7	2.95	0.6	0.62	10.79	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	7B	13.3	0.3	50	20	860	0.06	5.8	9	2.63	0.57	0.67	23.44	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	9C	7.7	0.1	50	20	1050	0.28	5.8	8	2.77	0.59	0.61	13.01	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	10C	8.8	0.1	50	20	970	0.06	5.8	11	2.39	0.55	0.57	11.99	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	11C	11.8	0.8	50	20	1890	0.008	5.8	18	1.9	0.5	0.82	18.38	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	12C	2.2	0.1	50	20	1310	0.024	5.8	16	2	0.52	0.54	2.38	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	13C	7.5	0.8	50	20	2500	0.007	5.8	22	1.72	0.48	0.82	10.58	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

APPENDIX 4

LACDPW MORA CALCULATIONS: EXISTING CONDITION

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4LADEPTH.RDT
5   300  100A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  111A  HYDROGRAPH AT OUTLET BASIN 100
6   300  100A  20  032.720A292 1400 00710                    G1
6   300  101A  20  0    99A29
6   300  102A  20  0    99A29
6   300  103A  20  0    99A29
6   300  104A  20  0    99A29
6   300  105A  20  0    99A29
6   300  106A  20  0    99A29
6   300  107A  20  0    99A29
6   300  108A  20  0    99A29
6   300  109A  20  0    99A29
6   300  110A  20  049.622A29
6   300  111A  20  0    99A29                             1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	100A	33.	30.	33.	30.	2	1400.	0.00710	0.00	0.00	0.	20	20	A29	0.00
300	101A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	102A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	103A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	104A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	105A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	106A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	107A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	108A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	109A	0.	0.	33.	25.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	110A	50.	42.	83.	60.	0	0.	0.00000	0.00	0.00	0.	20	22	A29	0.00
300	111A	0.	0.	83.	60.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 100

HYDROGRAPH AT 300 111A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	2.	800	2.	900	2.
1000	2.	1050	3.	1100	5.	1110	6.	1120	8.
1130	11.	1131	11.	1132	12.	1133	12.	1134	13.
1135	13.	1136	13.	1137	14.	1138	15.	1139	15.
1140	16.	1141	17.	1142	18.	1143	19.	1144	20.
1145	21.	1146	22.	1147	24.	1148	25.	1149	30.
1150	34.	1151	39.	1152	43.	1153	48.	1154	49.
1155	50.	1156	51.	1157	53.	1158	54.	1159	56.
1160	57.	1161	58.	1162	59.	1163	60.	1164	60.
1165	60.	1166	60.	1167	59.	1168	58.	1169	56.
1170	55.	1171	50.	1172	45.	1173	41.	1174	36.
1175	31.	1176	29.	1177	27.	1178	25.	1179	23.
1180	22.	1181	20.	1182	18.	1183	17.	1184	16.
1185	14.	1186	13.	1187	12.	1188	12.	1189	11.
1190	10.	1191	9.	1192	9.	1193	8.	1194	8.
1195	7.	1196	7.	1197	7.	1198	6.	1199	6.
1200	6.	1201	6.	1202	5.	1203	5.	1204	5.
1205	5.	1206	5.	1207	4.	1208	4.	1209	4.
1210	4.	1211	4.	1212	4.	1213	4.	1214	4.
1215	3.	1216	3.	1217	3.	1218	3.	1219	3.
1220	3.	1221	3.	1222	3.	1223	3.	1224	3.
1225	3.	1226	3.	1227	3.	1228	3.	1229	3.
1230	3.	1231	3.	1232	3.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	2.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	0.	1500	0.

Total Runoff = 5.870 Acre-Ft.
Peak Q = 60 CFS
Time to Peak Q = 1163 Minutes

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4LADEPTH.RDT
5   300  200A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  211A  HYDROGRAPH AT OUTLET BASIN 200
6   300  200A  20  017.317A292  800 01130                    G1
6   300  201A  20  0    99A29
6   300  202A  20  0    99A29
6   300  203A  20  0    99A29
6   300  204A  20  0    99A29
6   300  205A  20  0    99A29
6   300  206A  20  0    99A29
6   300  207A  20  0    99A29
6   300  208A  20  0    99A29
6   300  209A  20  0    99A29
6   300  210A  20  035.825A29
6   300  211A  20  0    99A29                             1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300 200A	17.	17.	17.	17.	2	800.	0.01130	0.00	0.00	0.	20	17	A29	0.00
300 201A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 202A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 203A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 204A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 205A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 206A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 207A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 208A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 209A	0.	0.	17.	16.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300 210A	36.	28.	53.	43.	0	0.	0.00000	0.00	0.00	0.	20	25	A29	0.00
300 211A	0.	0.	53.	43.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 200

HYDROGRAPH AT 300 211A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	1.
1000	2.	1050	2.	1100	3.	1110	4.	1120	5.
1130	8.	1131	8.	1132	8.	1133	8.	1134	9.
1135	9.	1136	9.	1137	10.	1138	10.	1139	11.
1140	11.	1141	12.	1142	12.	1143	13.	1144	14.
1145	15.	1146	16.	1147	16.	1148	17.	1149	20.
1150	23.	1151	26.	1152	30.	1153	33.	1154	35.
1155	37.	1156	38.	1157	40.	1158	41.	1159	42.
1160	42.	1161	43.	1162	42.	1163	42.	1164	42.
1165	41.	1166	40.	1167	39.	1168	38.	1169	37.
1170	35.	1171	33.	1172	30.	1173	28.	1174	24.
1175	20.	1176	16.	1177	13.	1178	10.	1179	8.
1180	7.	1181	6.	1182	6.	1183	5.	1184	5.
1185	5.	1186	4.	1187	4.	1188	4.	1189	4.
1190	3.	1191	3.	1192	3.	1193	3.	1194	3.
1195	3.	1196	3.	1197	3.	1198	3.	1199	2.
1200	2.	1201	2.	1202	2.	1203	2.	1204	2.
1205	2.	1206	2.	1207	2.	1208	2.	1209	2.
1210	2.	1211	2.	1212	2.	1213	2.	1214	2.
1215	2.	1216	2.	1217	2.	1218	2.	1219	2.
1220	2.	1221	2.	1222	2.	1223	2.	1224	1.
1225	1.	1226	1.	1227	1.	1228	1.	1229	1.
1230	1.	1231	1.	1232	1.	1233	1.	1234	1.
1235	1.	1236	1.	1237	1.	1238	1.	1239	1.
1240	1.	1241	1.	1242	1.	1243	1.	1244	1.
1245	1.	1246	1.	1247	1.	1248	1.	1249	1.
1250	1.	1251	1.	1252	1.	1253	1.	1254	1.
1255	1.	1256	1.	1257	1.	1258	1.	1259	1.
1260	1.	1261	1.	1262	1.	1263	1.	1264	1.
1265	1.	1266	1.	1267	1.	1268	1.	1269	1.
1270	1.	1271	1.	1272	1.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	0.	1500	0.

Total Runoff = 4.027 Acre-Ft.
Peak Q = 43 CFS
Time to Peak Q = 1161 Minutes

```
4LADEPTH.RDT
5   300  400A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  426A  HYDROGRAPH AT OUTLET BASIN 400
6   300  400B  20  018.424A294  155 01300                    G1
6   300  401B  20  0    99A29
6   300  402B  20  0    99A29
6   300  403B  20  0    99A29
6   300  404B  20  0    99A29
6   300  405B  20  038.929A292  920 00500
6   300  406B  20  0    99A29
6   300  407B  20  0    99A29
6   300  408C  20  015.3 8A292 1990 04800
6   300  409C  20  0    99A29
6   300  410C  20  044.319A29
6   300  411BC 20  0      A29
6   300  412B  20  0    99A29
6   300  413B  20  0    99A29
6   300  414B  20  0    99A29
6   300  415B  20  035.311A292 2430 01400
6   300  416B  20  0    99A29
6   300  417B  20  0    99A29
6   300  418B  20  0    99A29
6   300  419B  20  0    99A29
6   300  420A  20  034.425A293  600 01000
6   300  421AB 20  0      A29
6   300  422A  20  0    99A29
6   300  423A  20  0    99A29
6   300  424A  20  0    99A29
6   300  425A  20  039.921A29
6   300  426A  20  0    99A29                              1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	400B	18.	14.	18.	14.	4	155.	0.01300	2.00	0.00	0.	20	24	A29	0.00
300	401B	0.	0.	18.	14.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	402B	0.	0.	18.	14.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	403B	0.	0.	18.	14.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	404B	0.	0.	18.	14.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	405B	39.	27.	57.	41.	2	920.	0.00500	0.00	0.00	0.	20	29	A29	0.00
300	406B	0.	0.	57.	39.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	407B	0.	0.	57.	39.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	408C	15.	24.	15.	24.	2	1990.	0.04800	0.00	0.00	0.	20	8	A29	0.00
300	409C	0.	0.	15.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	410C	44.	41.	59.	58.	0	0.	0.00000	0.00	0.00	0.	20	19	A29	0.00
300	411BC	59.	58.	116.	91.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
300	412B	0.	0.	116.	91.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	413B	0.	0.	116.	91.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	414B	0.	0.	116.	91.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	415B	35.	47.	151.	127.	2	2430.	0.01400	0.00	0.00	0.	20	11	A29	0.00
300	416B	0.	0.	151.	108.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	417B	0.	0.	151.	108.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	418B	0.	0.	151.	108.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	419B	0.	0.	151.	108.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	420A	34.	26.	34.	26.	3	600.	0.01000	0.00	0.00	0.	20	25	A29	0.00
300	421AB	151.	108.	185.	133.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
300	422A	0.	0.	185.	133.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	423A	0.	0.	185.	133.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	424A	0.	0.	185.	133.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	425A	40.	35.	225.	161.	0	0.	0.00000	0.00	0.00	0.	20	21	A29	0.00
300	426A	0.	0.	225.	161.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 400

HYDROGRAPH AT 300 426A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	4.	400	4.
500	4.	600	4.	700	4.	800	5.	900	5.
1000	6.	1050	8.	1100	13.	1110	14.	1120	17.
1130	24.	1131	24.	1132	25.	1133	26.	1134	27.
1135	28.	1136	29.	1137	30.	1138	31.	1139	33.
1140	34.	1141	35.	1142	37.	1143	39.	1144	40.
1145	42.	1146	45.	1147	47.	1148	50.	1149	56.
1150	61.	1151	67.	1152	74.	1153	82.	1154	88.
1155	94.	1156	102.	1157	110.	1158	118.	1159	127.
1160	134.	1161	141.	1162	148.	1163	153.	1164	157.
1165	160.	1166	161.	1167	160.	1168	158.	1169	155.
1170	148.	1171	141.	1172	134.	1173	126.	1174	119.
1175	113.	1176	107.	1177	100.	1178	93.	1179	87.
1180	81.	1181	76.	1182	72.	1183	68.	1184	64.
1185	61.	1186	58.	1187	56.	1188	53.	1189	51.
1190	49.	1191	47.	1192	45.	1193	43.	1194	41.
1195	39.	1196	37.	1197	35.	1198	33.	1199	32.
1200	30.	1201	28.	1202	27.	1203	26.	1204	24.
1205	23.	1206	22.	1207	21.	1208	20.	1209	19.
1210	18.	1211	18.	1212	17.	1213	16.	1214	16.
1215	15.	1216	14.	1217	14.	1218	13.	1219	13.
1220	12.	1221	12.	1222	12.	1223	11.	1224	11.
1225	11.	1226	10.	1227	10.	1228	10.	1229	10.
1230	9.	1231	9.	1232	9.	1233	9.	1234	9.
1235	8.	1236	8.	1237	8.	1238	8.	1239	8.
1240	8.	1241	8.	1242	7.	1243	7.	1244	7.
1245	7.	1246	7.	1247	7.	1248	7.	1249	7.
1250	7.	1251	7.	1252	6.	1253	6.	1254	6.
1255	6.	1256	6.	1257	6.	1258	6.	1259	6.
1260	6.	1261	6.	1262	6.	1263	6.	1264	6.
1265	6.	1266	6.	1267	6.	1268	6.	1269	6.
1270	5.	1271	5.	1272	5.	1273	5.	1274	5.
1275	5.	1276	5.	1277	5.	1278	5.	1279	5.
1280	5.	1281	5.	1282	5.	1283	5.	1284	5.
1285	5.	1286	5.	1287	5.	1288	5.	1289	5.
1290	5.	1291	5.	1292	5.	1293	5.	1294	5.
1295	5.	1296	5.	1297	5.	1298	5.	1299	5.
1300	5.	1310	4.	1320	4.	1330	4.	1340	4.
1350	4.	1360	4.	1370	4.	1380	4.	1390	4.
1400	4.	1420	4.	1440	3.	1460	3.	1500	3.

Total Runoff = 16.926 Acre-Ft.
Peak Q = 161 CFS
Time to Peak Q = 1166 Minutes

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4LADEPTH.RDT
5   300  500A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  510A  HYDROGRAPH AT OUTLET BASIN 500
6   300  500A  20  026.520A292 1530 00590                    G1
6   300  501A  20  0    99A29
6   300  502A  20  0    99A29
6   300  503A  20  0    99A29
6   300  504A  20  0    99A29
6   300  505A  20  0    99A29
6   300  506A  20  0    99A29
6   300  507A  20  0    99A29
6   300  508A  20  0    99A29
6   300  509A  20  0    99A29
6   300  510A  20  040.025A29                             1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
300	500A	26.	23.	26.	23.	2	1530.	0.00590	0.00	0.00	0.	20	20	A29	0.00
300	501A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	502A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	503A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	504A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	505A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	506A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	507A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	508A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	509A	0.	0.	26.	19.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
300	510A	40.	31.	66.	43.	0	0.	0.00000	0.00	0.00	0.	20	25	A29	0.00

HYDROGRAPH AT OUTLET BASIN 500

HYDROGRAPH AT 300 510A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	2.
1000	2.	1050	3.	1100	4.	1110	4.	1120	6.
1130	8.	1131	8.	1132	9.	1133	9.	1134	9.
1135	10.	1136	10.	1137	10.	1138	11.	1139	11.
1140	12.	1141	12.	1142	13.	1143	14.	1144	14.
1145	15.	1146	16.	1147	17.	1148	18.	1149	21.
1150	25.	1151	28.	1152	31.	1153	34.	1154	35.
1155	36.	1156	37.	1157	37.	1158	38.	1159	38.
1160	39.	1161	40.	1162	41.	1163	42.	1164	43.
1165	43.	1166	43.	1167	43.	1168	43.	1169	43.
1170	42.	1171	41.	1172	40.	1173	39.	1174	36.
1175	33.	1176	29.	1177	26.	1178	23.	1179	21.
1180	20.	1181	19.	1182	17.	1183	16.	1184	15.
1185	14.	1186	13.	1187	13.	1188	12.	1189	11.
1190	10.	1191	10.	1192	9.	1193	9.	1194	8.
1195	8.	1196	7.	1197	7.	1198	7.	1199	6.
1200	6.	1201	6.	1202	5.	1203	5.	1204	5.
1205	5.	1206	5.	1207	4.	1208	4.	1209	4.
1210	4.	1211	4.	1212	4.	1213	4.	1214	3.
1215	3.	1216	3.	1217	3.	1218	3.	1219	3.
1220	3.	1221	3.	1222	3.	1223	3.	1224	3.
1225	3.	1226	3.	1227	3.	1228	2.	1229	2.
1230	2.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	0.

Total Runoff = 4.859 Acre-Ft.
Peak Q = 43 CFS
Time to Peak Q = 1164 Minutes

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4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Existing
5 50300   17A  HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site
6 50300    1A 020  0   6 8A292  890 26500                      G1
6 50300    2A 020  0   4 6A292  200 01000
6 50300    3A  20  0   2 5A294  275 01000
6 50300    4A  20  0  1210A294  385 02000
6 50300    5B  20  0   4 5A292  530 32100
6 50300    6B  20  0  2515A294  835 02200
6 50300    7AB 20  0      A29
6 50300    8A  20  0   2 5A294  320 02000
6 50300    9A  20  0   3 5A294  900 01000
6 50300   10C  20  0  3214A292 1750 01500
6 50300   11C  20  0  1611A292 2800 01000
6 50300   12AC020         A29
6 50300   13D  20 65  1017A292 2370 01800
6 50300   14D  20 65  1210A292 1250 03200
6 50300   15AD 20         A29
6 50300   16A  20  0  1223A29
6 50300   17A 020  0    99A29                            1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Existing

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM RAIN ZONE	DAY 4 PCT IMPV
50300	1A	6.	10.	6.	10.	2	890.	0.26500	0.00	0.00	0.	20	8	A29	0.00
50300	2A	4.	8.	10.	17.	2	200.	0.01000	0.00	0.00	0.	20	6	A29	0.00
50300	3A	2.	4.	12.	18.	4	275.	0.01000	2.00	0.00	0.	20	5	A29	0.00
50300	4A	12.	17.	24.	34.	4	385.	0.02000	2.00	0.00	0.	20	10	A29	0.00
50300	5B	4.	9.	4.	9.	2	530.	0.32100	0.00	0.00	0.	20	5	A29	0.00
50300	6B	25.	27.	29.	35.	4	835.	0.02200	2.00	0.00	0.	20	15	A29	0.00
50300	7AB	29.	34.	53.	68.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	8A	2.	4.	55.	69.	4	320.	0.02000	2.75	0.00	0.	20	5	A29	0.00
50300	9A	3.	7.	58.	72.	4	900.	0.01000	3.00	0.00	0.	20	5	A29	0.00
50300	10C	32.	36.	32.	36.	2	1750.	0.01500	0.00	0.00	0.	20	14	A29	0.00
50300	11C	16.	21.	48.	40.	2	2800.	0.01000	0.00	0.00	0.	20	11	A29	0.00
50300	12AC	48.	29.	106.	78.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	13D	10.	16.	10.	16.	2	2370.	0.01800	0.00	0.00	0.	20	17	A29	0.65
50300	14D	12.	25.	22.	32.	2	1250.	0.03200	0.00	0.00	0.	20	10	A29	0.65
50300	15AD	22.	30.	128.	107.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	16A	12.	10.	140.	117.	0	0.	0.00000	0.00	0.00	0.	20	23	A29	0.00
50300	17A	0.	0.	140.	117.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site

HYDROGRAPH AT 50300 17A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	4.	200	4.	300	4.	400	5.
500	5.	600	5.	700	6.	800	6.	900	7.
1000	8.	1050	10.	1100	13.	1110	15.	1120	18.
1130	23.	1131	23.	1132	24.	1133	25.	1134	25.
1135	26.	1136	27.	1137	27.	1138	28.	1139	29.
1140	30.	1141	32.	1142	33.	1143	34.	1144	36.
1145	37.	1146	39.	1147	41.	1148	44.	1149	47.
1150	51.	1151	56.	1152	63.	1153	72.	1154	82.
1155	93.	1156	102.	1157	109.	1158	114.	1159	116.
1160	117.	1161	116.	1162	113.	1163	107.	1164	101.
1165	94.	1166	89.	1167	84.	1168	80.	1169	76.
1170	73.	1171	70.	1172	67.	1173	63.	1174	61.
1175	58.	1176	57.	1177	55.	1178	54.	1179	53.
1180	52.	1181	51.	1182	50.	1183	48.	1184	47.
1185	46.	1186	44.	1187	43.	1188	41.	1189	40.
1190	38.	1191	37.	1192	35.	1193	34.	1194	33.
1195	32.	1196	30.	1197	29.	1198	28.	1199	27.
1200	26.	1201	25.	1202	24.	1203	23.	1204	23.
1205	22.	1206	21.	1207	21.	1208	20.	1209	19.
1210	19.	1211	18.	1212	18.	1213	17.	1214	17.
1215	16.	1216	16.	1217	15.	1218	15.	1219	15.
1220	14.	1221	14.	1222	14.	1223	13.	1224	13.
1225	13.	1226	13.	1227	12.	1228	12.	1229	12.
1230	12.	1231	11.	1232	11.	1233	11.	1234	11.
1235	11.	1236	11.	1237	10.	1238	10.	1239	10.
1240	10.	1241	10.	1242	10.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	9.	1249	9.
1250	9.	1251	9.	1252	9.	1253	8.	1254	8.
1255	8.	1256	8.	1257	8.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	8.
1265	8.	1266	7.	1267	7.	1268	7.	1269	7.
1270	7.	1271	7.	1272	7.	1273	7.	1274	7.
1275	7.	1276	7.	1277	7.	1278	7.	1279	7.
1280	7.	1281	7.	1282	7.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	6.	1289	6.
1290	6.	1291	6.	1292	6.	1293	6.	1294	6.
1295	6.	1296	6.	1297	6.	1298	6.	1299	6.
1300	6.	1310	6.	1320	6.	1330	5.	1340	5.
1350	5.	1360	5.	1370	5.	1380	5.	1390	5.
1400	5.	1420	5.	1440	4.	1460	4.	1500	4.

Total Runoff = 17.108 Acre-Ft.
Peak Q = 117 CFS
Time to Peak Q = 1160 Minutes

APPENDIX 5

LACDPW MORA CALCULATIONS: EXISTING CONDITION - BURNED

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4LADEPTH.RDT
5   300  100A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  111A  HYDROGRAPH AT OUTLET BASIN 100 - BURNED
6   300  100A 198  032.720A292 1400 00710                     G1
6   300  101A 198  0    99A29
6   300  102A 198  0    99A29
6   300  103A 198  0    99A29
6   300  104A 198  0    99A29
6   300  105A 198  0    99A29
6   300  106A 198  0    99A29
6   300  107A 198  0    99A29
6   300  108A 198  0    99A29
6   300  109A 198  0    99A29
6   300  110A 198  049.622A29
6   300  111A 198  0    99A29                              1  2
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10-20-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	100A	33.	41.	33.	41.	2	1400.	0.00710	0.00	0.00	0.	198	20	A29	0.00
300	101A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	102A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	103A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	104A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	105A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	106A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	107A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	108A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	109A	0.	0.	33.	37.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	110A	50.	59.	83.	88.	0	0.	0.00000	0.00	0.00	0.	198	22	A29	0.00
300	111A	0.	0.	83.	88.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 100 - BURNED

HYDROGRAPH AT 300 111A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	5.	200	6.	300	6.	400	6.
500	7.	600	7.	700	8.	800	9.	900	10.
1000	12.	1050	15.	1100	19.	1110	21.	1120	23.
1130	27.	1131	27.	1132	28.	1133	28.	1134	29.
1135	29.	1136	29.	1137	30.	1138	31.	1139	31.
1140	32.	1141	33.	1142	35.	1143	36.	1144	37.
1145	38.	1146	40.	1147	42.	1148	43.	1149	49.
1150	54.	1151	60.	1152	66.	1153	72.	1154	74.
1155	75.	1156	77.	1157	79.	1158	81.	1159	83.
1160	85.	1161	86.	1162	87.	1163	88.	1164	88.
1165	87.	1166	87.	1167	85.	1168	84.	1169	82.
1170	80.	1171	74.	1172	67.	1173	61.	1174	55.
1175	50.	1176	48.	1177	45.	1178	42.	1179	40.
1180	37.	1181	35.	1182	33.	1183	31.	1184	29.
1185	28.	1186	27.	1187	25.	1188	24.	1189	24.
1190	23.	1191	22.	1192	21.	1193	20.	1194	20.
1195	19.	1196	19.	1197	18.	1198	18.	1199	17.
1200	17.	1201	17.	1202	16.	1203	16.	1204	16.
1205	16.	1206	15.	1207	15.	1208	15.	1209	15.
1210	14.	1211	14.	1212	14.	1213	14.	1214	14.
1215	13.	1216	13.	1217	13.	1218	13.	1219	13.
1220	13.	1221	13.	1222	12.	1223	12.	1224	12.
1225	12.	1226	12.	1227	12.	1228	12.	1229	12.
1230	11.	1231	11.	1232	11.	1233	11.	1234	11.
1235	11.	1236	11.	1237	11.	1238	11.	1239	11.
1240	11.	1241	11.	1242	10.	1243	10.	1244	10.
1245	10.	1246	10.	1247	10.	1248	10.	1249	10.
1250	10.	1251	10.	1252	10.	1253	10.	1254	10.
1255	10.	1256	10.	1257	10.	1258	9.	1259	9.
1260	9.	1261	9.	1262	9.	1263	9.	1264	9.
1265	9.	1266	9.	1267	9.	1268	9.	1269	9.
1270	9.	1271	9.	1272	9.	1273	9.	1274	9.
1275	9.	1276	9.	1277	9.	1278	9.	1279	9.
1280	9.	1281	9.	1282	8.	1283	8.	1284	8.
1285	8.	1286	8.	1287	8.	1288	8.	1289	8.
1290	8.	1291	8.	1292	8.	1293	8.	1294	8.
1295	8.	1296	8.	1297	8.	1298	8.	1299	8.
1300	8.	1310	8.	1320	7.	1330	7.	1340	7.
1350	7.	1360	7.	1370	6.	1380	6.	1390	6.
1400	6.	1420	6.	1440	6.	1460	2.	1500	2.

Total Runoff = 20.163 Acre-Ft.
Peak Q = 88 CFS
Time to Peak Q = 1163 Minutes

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4LADEPTH.RDT
5   300  200A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  211A  HYDROGRAPH AT OUTLET BASIN 200 - BURNED
6   300  200A 198  017.317A292  800 01130                    G1
6   300  201A 198  0    99A29
6   300  202A 198  0    99A29
6   300  203A 198  0    99A29
6   300  204A 198  0    99A29
6   300  205A 198  0    99A29
6   300  206A 198  0    99A29
6   300  207A 198  0    99A29
6   300  208A 198  0    99A29
6   300  209A 198  0    99A29
6   300  210A 198  035.825A29
6   300  211A 198  0    99A29                             1  2
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10-20-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION	SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300 200A	17.	23.	17.	23.	2	800.	0.01130	0.00	0.00	0.	198	17	A29	0.00
300 201A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 202A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 203A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 204A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 205A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 206A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 207A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 208A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 209A	0.	0.	17.	22.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300 210A	36.	39.	53.	60.	0	0.	0.00000	0.00	0.00	0.	198	25	A29	0.00
300 211A	0.	0.	53.	60.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 200 - BURNED

HYDROGRAPH AT 300 211A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	4.	200	4.	300	4.	400	4.
500	4.	600	5.	700	5.	800	6.	900	6.
1000	8.	1050	10.	1100	12.	1110	13.	1120	15.
1130	18.	1131	18.	1132	18.	1133	19.	1134	19.
1135	19.	1136	19.	1137	20.	1138	20.	1139	21.
1140	21.	1141	22.	1142	23.	1143	23.	1144	24.
1145	26.	1146	27.	1147	28.	1148	29.	1149	33.
1150	36.	1151	40.	1152	44.	1153	49.	1154	51.
1155	53.	1156	56.	1157	57.	1158	59.	1159	60.
1160	60.	1161	60.	1162	60.	1163	60.	1164	59.
1165	58.	1166	57.	1167	55.	1168	54.	1169	52.
1170	49.	1171	47.	1172	44.	1173	41.	1174	36.
1175	31.	1176	26.	1177	22.	1178	19.	1179	18.
1180	17.	1181	16.	1182	15.	1183	14.	1184	14.
1185	13.	1186	13.	1187	13.	1188	12.	1189	12.
1190	12.	1191	11.	1192	11.	1193	11.	1194	11.
1195	10.	1196	10.	1197	10.	1198	10.	1199	10.
1200	10.	1201	9.	1202	9.	1203	9.	1204	9.
1205	9.	1206	9.	1207	9.	1208	9.	1209	9.
1210	8.	1211	8.	1212	8.	1213	8.	1214	8.
1215	8.	1216	8.	1217	8.	1218	8.	1219	8.
1220	8.	1221	8.	1222	7.	1223	7.	1224	7.
1225	7.	1226	7.	1227	7.	1228	7.	1229	7.
1230	7.	1231	7.	1232	7.	1233	7.	1234	7.
1235	7.	1236	7.	1237	7.	1238	7.	1239	7.
1240	7.	1241	7.	1242	6.	1243	6.	1244	6.
1245	6.	1246	6.	1247	6.	1248	6.	1249	6.
1250	6.	1251	6.	1252	6.	1253	6.	1254	6.
1255	6.	1256	6.	1257	6.	1258	6.	1259	6.
1260	6.	1261	6.	1262	6.	1263	6.	1264	6.
1265	6.	1266	6.	1267	6.	1268	6.	1269	6.
1270	6.	1271	6.	1272	6.	1273	6.	1274	5.
1275	5.	1276	5.	1277	5.	1278	5.	1279	5.
1280	5.	1281	5.	1282	5.	1283	5.	1284	5.
1285	5.	1286	5.	1287	5.	1288	5.	1289	5.
1290	5.	1291	5.	1292	5.	1293	5.	1294	5.
1295	5.	1296	5.	1297	5.	1298	5.	1299	5.
1300	5.	1310	5.	1320	5.	1330	5.	1340	4.
1350	4.	1360	4.	1370	4.	1380	4.	1390	4.
1400	4.	1420	4.	1440	4.	1460	2.	1500	1.

Total Runoff = 13.075 Acre-Ft.
Peak Q = 60 CFS
Time to Peak Q = 1159 Minutes

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4LADEPTH.RDT
5   300  400A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  426A  HYDROGRAPH AT OUTLET BASIN 400 - BURNED
6   300  400B 198  018.424A294  155 01300                    G1
6   300  401B 198  0    99A29
6   300  402B 198  0    99A29
6   300  403B 198  0    99A29
6   300  404B 198  0    99A29
6   300  405B 198  038.929A292  920 00500
6   300  406B 198  0    99A29
6   300  407B 198  0    99A29
6   300  408C 198  015.3 8A292 1990 04800
6   300  409C 198  0    99A29
6   300  410C 198  044.319A29
6   300  411BC198  0      A29
6   300  412B 198  0    99A29
6   300  413B 198  0    99A29
6   300  414B 198  0    99A29
6   300  415B 198  035.311A292 2430 01400
6   300  416B 198  0    99A29
6   300  417B 198  0    99A29
6   300  418B 198  0    99A29
6   300  419B 198  0    99A29
6   300  420A 198  034.425A293  600 01000
6   300  421AB198  0      A29
6   300  422A 198  0    99A29
6   300  423A 198  0    99A29
6   300  424A 198  0    99A29
6   300  425A 198  039.921A29
6   300  426A 198  0    99A29                            1  2
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10-20-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	400B	18.	20.	18.	20.	4	155.	0.01300	2.00	0.00	0.	198	24	A29	0.00
300	401B	0.	0.	18.	20.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	402B	0.	0.	18.	20.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	403B	0.	0.	18.	20.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	404B	0.	0.	18.	20.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	405B	39.	39.	57.	59.	2	920.	0.00500	0.00	0.00	0.	198	29	A29	0.00
300	406B	0.	0.	57.	56.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	407B	0.	0.	57.	56.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	408C	15.	31.	15.	31.	2	1990.	0.04800	0.00	0.00	0.	198	8	A29	0.00
300	409C	0.	0.	15.	26.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	410C	44.	56.	59.	81.	0	0.	0.00000	0.00	0.00	0.	198	19	A29	0.00
300	411BC	59.	81.	116.	130.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
300	412B	0.	0.	116.	130.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	413B	0.	0.	116.	130.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	414B	0.	0.	116.	130.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	415B	35.	62.	151.	182.	2	2430.	0.01400	0.00	0.00	0.	198	11	A29	0.00
300	416B	0.	0.	151.	162.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	417B	0.	0.	151.	162.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	418B	0.	0.	151.	162.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	419B	0.	0.	151.	162.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	420A	34.	37.	34.	37.	3	600.	0.01000	0.00	0.00	0.	198	25	A29	0.00
300	421AB	151.	162.	185.	197.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
300	422A	0.	0.	185.	197.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	423A	0.	0.	185.	197.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	424A	0.	0.	185.	197.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	425A	40.	48.	225.	239.	0	0.	0.00000	0.00	0.00	0.	198	21	A29	0.00
300	426A	0.	0.	225.	239.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 400 - BURNED

HYDROGRAPH AT 300 426A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	15.	200	15.	300	16.	400	17.
500	18.	600	20.	700	21.	800	24.	900	27.
1000	32.	1050	39.	1100	50.	1110	53.	1120	59.
1130	68.	1131	69.	1132	69.	1133	70.	1134	71.
1135	73.	1136	74.	1137	75.	1138	76.	1139	78.
1140	79.	1141	81.	1142	83.	1143	85.	1144	87.
1145	90.	1146	93.	1147	96.	1148	100.	1149	106.
1150	114.	1151	122.	1152	131.	1153	141.	1154	149.
1155	159.	1156	170.	1157	181.	1158	192.	1159	202.
1160	212.	1161	220.	1162	227.	1163	234.	1164	238.
1165	239.	1166	238.	1167	235.	1168	230.	1169	224.
1170	214.	1171	204.	1172	194.	1173	184.	1174	174.
1175	166.	1176	158.	1177	149.	1178	141.	1179	133.
1180	126.	1181	120.	1182	115.	1183	110.	1184	106.
1185	102.	1186	99.	1187	95.	1188	92.	1189	89.
1190	86.	1191	83.	1192	80.	1193	77.	1194	74.
1195	71.	1196	69.	1197	66.	1198	64.	1199	62.
1200	60.	1201	58.	1202	56.	1203	55.	1204	53.
1205	52.	1206	50.	1207	49.	1208	48.	1209	47.
1210	46.	1211	45.	1212	44.	1213	43.	1214	42.
1215	41.	1216	41.	1217	40.	1218	39.	1219	39.
1220	38.	1221	38.	1222	37.	1223	37.	1224	36.
1225	36.	1226	35.	1227	35.	1228	35.	1229	34.
1230	34.	1231	33.	1232	33.	1233	33.	1234	32.
1235	32.	1236	32.	1237	32.	1238	31.	1239	31.
1240	31.	1241	30.	1242	30.	1243	30.	1244	30.
1245	30.	1246	29.	1247	29.	1248	29.	1249	29.
1250	28.	1251	28.	1252	28.	1253	28.	1254	28.
1255	28.	1256	27.	1257	27.	1258	27.	1259	27.
1260	27.	1261	27.	1262	26.	1263	26.	1264	26.
1265	26.	1266	26.	1267	26.	1268	26.	1269	25.
1270	25.	1271	25.	1272	25.	1273	25.	1274	25.
1275	25.	1276	24.	1277	24.	1278	24.	1279	24.
1280	24.	1281	24.	1282	24.	1283	24.	1284	24.
1285	23.	1286	23.	1287	23.	1288	23.	1289	23.
1290	23.	1291	23.	1292	23.	1293	23.	1294	23.
1295	22.	1296	22.	1297	22.	1298	22.	1299	22.
1300	22.	1310	21.	1320	21.	1330	20.	1340	19.
1350	19.	1360	18.	1370	18.	1380	17.	1390	17.
1400	17.	1420	16.	1440	15.	1460	12.	1500	12.

Total Runoff = 55.135 Acre-Ft.
Peak Q = 239 CFS
Time to Peak Q = 1165 Minutes

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4LADEPTH.RDT
5   300  500A  50 YR River Village - Existing Condition - Newhall Ranch
5   300  510A  HYDROGRAPH AT OUTLET BASIN 500 - BURNED
6   300  500A 198  026.520A292 1530 00590                      G1
6   300  501A 198  0    99A29
6   300  502A 198  0    99A29
6   300  503A 198  0    99A29
6   300  504A 198  0    99A29
6   300  505A 198  0    99A29
6   300  506A 198  0    99A29
6   300  507A 198  0    99A29
6   300  508A 198  0    99A29
6   300  509A 198  0    99A29
6   300  510A 198  040.025A29                              1  2
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10-20-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Existing Condition - Newhall Ranch STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	500A	26.	32.	26.	32.	2	1530.	0.00590	0.00	0.00	0.	198	20	A29	0.00
300	501A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	502A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	503A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	504A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	505A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	506A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	507A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	508A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	509A	0.	0.	26.	28.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00
300	510A	40.	44.	66.	64.	0	0.	0.00000	0.00	0.00	0.	198	25	A29	0.00

HYDROGRAPH AT OUTLET BASIN 500 - BURNED

HYDROGRAPH AT 300 510A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	4.	200	5.	300	5.	400	5.
500	5.	600	6.	700	6.	800	7.	900	8.
1000	10.	1050	12.	1100	15.	1110	16.	1120	18.
1130	21.	1131	21.	1132	21.	1133	22.	1134	22.
1135	22.	1136	23.	1137	23.	1138	24.	1139	24.
1140	24.	1141	25.	1142	26.	1143	27.	1144	28.
1145	29.	1146	30.	1147	31.	1148	32.	1149	36.
1150	40.	1151	44.	1152	48.	1153	52.	1154	54.
1155	55.	1156	55.	1157	56.	1158	57.	1159	59.
1160	60.	1161	61.	1162	62.	1163	63.	1164	64.
1165	64.	1166	64.	1167	64.	1168	64.	1169	63.
1170	62.	1171	61.	1172	60.	1173	58.	1174	54.
1175	50.	1176	45.	1177	41.	1178	38.	1179	36.
1180	34.	1181	32.	1182	30.	1183	29.	1184	27.
1185	26.	1186	25.	1187	23.	1188	22.	1189	22.
1190	21.	1191	20.	1192	19.	1193	19.	1194	18.
1195	17.	1196	17.	1197	16.	1198	16.	1199	15.
1200	15.	1201	15.	1202	14.	1203	14.	1204	14.
1205	14.	1206	13.	1207	13.	1208	13.	1209	13.
1210	12.	1211	12.	1212	12.	1213	12.	1214	12.
1215	11.	1216	11.	1217	11.	1218	11.	1219	11.
1220	11.	1221	10.	1222	10.	1223	10.	1224	10.
1225	10.	1226	10.	1227	10.	1228	10.	1229	10.
1230	10.	1231	9.	1232	9.	1233	9.	1234	9.
1235	9.	1236	9.	1237	9.	1238	9.	1239	9.
1240	9.	1241	9.	1242	9.	1243	9.	1244	8.
1245	8.	1246	8.	1247	8.	1248	8.	1249	8.
1250	8.	1251	8.	1252	8.	1253	8.	1254	8.
1255	8.	1256	8.	1257	8.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	7.
1265	7.	1266	7.	1267	7.	1268	7.	1269	7.
1270	7.	1271	7.	1272	7.	1273	7.	1274	7.
1275	7.	1276	7.	1277	7.	1278	7.	1279	7.
1280	7.	1281	7.	1282	7.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	7.	1289	7.
1290	7.	1291	7.	1292	7.	1293	7.	1294	6.
1295	6.	1296	6.	1297	6.	1298	6.	1299	6.
1300	6.	1310	6.	1320	6.	1330	6.	1340	6.
1350	5.	1360	5.	1370	5.	1380	5.	1390	5.
1400	5.	1420	5.	1440	4.	1460	2.	1500	2.

Total Runoff = 16.133 Acre-Ft.
Peak Q = 64 CFS
Time to Peak Q = 1164 Minutes

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4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Existing
5 50300   17A  HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site
6 50300    1A 198  0   6 8A292  890 26500                      G1
6 50300    2A 198  0   4 6A292  200 01000
6 50300    3A 198  0   2 5A294  275 01000
6 50300    4A 198  0  1210A294  385 02000
6 50300    5B 198  0   4 5A292  530 32100
6 50300    6B 198  0  2515A294  835 02200
6 50300    7AB198  0      A29
6 50300    8A 198  0   2 5A294  320 02000
6 50300    9A 198  0   3 5A294  900 01000
6 50300   10C 198  0  3214A292 1750 01500
6 50300   11C 198  0  1611A292 2800 01000
6 50300   12AC198         A29
6 50300   13D 198 65  1017A292 2370 01800
6 50300   14D 198 65  1210A292 1250 03200
6 50300   15AD198         A29
6 50300   16A 198  0  1223A29
6 50300   17A 198  0    99A29                            1  2
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10-19-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Existing

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
50300	1A	6.	13.	6.	13.	2	890.	0.26500	0.00	0.00	0.	198	8	A29	0.00
50300	2A	4.	10.	10.	21.	2	200.	0.01000	0.00	0.00	0.	198	6	A29	0.00
50300	3A	2.	5.	12.	24.	4	275.	0.01000	2.00	0.00	0.	198	5	A29	0.00
50300	4A	12.	22.	24.	45.	4	385.	0.02000	2.25	0.00	0.	198	10	A29	0.00
50300	5B	4.	11.	4.	11.	2	530.	0.32100	0.00	0.00	0.	198	5	A29	0.00
50300	6B	25.	37.	29.	47.	4	835.	0.02200	2.25	0.00	0.	198	15	A29	0.00
50300	7AB	29.	46.	53.	90.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	8A	2.	5.	55.	93.	4	320.	0.02000	3.00	0.00	0.	198	5	A29	0.00
50300	9A	3.	8.	58.	96.	4	900.	0.01000	3.50	0.00	0.	198	5	A29	0.00
50300	10C	32.	49.	32.	49.	2	1750.	0.01500	0.00	0.00	0.	198	14	A29	0.00
50300	11C	16.	28.	48.	59.	2	2800.	0.01000	0.00	0.00	0.	198	11	A29	0.00
50300	12AC	48.	46.	106.	115.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	13D	10.	17.	10.	17.	2	2370.	0.01800	0.00	0.00	0.	198	17	A29	0.65
50300	14D	12.	27.	22.	34.	2	1250.	0.03200	0.00	0.00	0.	198	10	A29	0.65
50300	15AD	22.	33.	128.	147.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	16A	12.	14.	140.	160.	0	0.	0.00000	0.00	0.00	0.	198	23	A29	0.00
50300	17A	0.	0.	140.	160.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site

HYDROGRAPH AT 50300 17A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	10.	200	11.	300	11.	400	12.
500	12.	600	13.	700	15.	800	16.	900	18.
1000	22.	1050	26.	1100	33.	1110	36.	1120	41.
1130	47.	1131	47.	1132	48.	1133	49.	1134	49.
1135	50.	1136	51.	1137	52.	1138	53.	1139	54.
1140	55.	1141	57.	1142	58.	1143	60.	1144	62.
1145	64.	1146	67.	1147	69.	1148	72.	1149	76.
1150	81.	1151	88.	1152	97.	1153	109.	1154	121.
1155	134.	1156	145.	1157	153.	1158	158.	1159	160.
1160	160.	1161	158.	1162	153.	1163	146.	1164	139.
1165	132.	1166	126.	1167	121.	1168	117.	1169	112.
1170	108.	1171	104.	1172	99.	1173	96.	1174	92.
1175	90.	1176	88.	1177	86.	1178	85.	1179	83.
1180	81.	1181	79.	1182	77.	1183	75.	1184	72.
1185	70.	1186	68.	1187	65.	1188	63.	1189	61.
1190	59.	1191	57.	1192	55.	1193	53.	1194	52.
1195	50.	1196	48.	1197	47.	1198	45.	1199	44.
1200	43.	1201	42.	1202	41.	1203	40.	1204	39.
1205	38.	1206	37.	1207	36.	1208	35.	1209	34.
1210	34.	1211	33.	1212	32.	1213	32.	1214	31.
1215	31.	1216	30.	1217	29.	1218	29.	1219	29.
1220	28.	1221	28.	1222	27.	1223	27.	1224	26.
1225	26.	1226	26.	1227	25.	1228	25.	1229	25.
1230	24.	1231	24.	1232	24.	1233	24.	1234	23.
1235	23.	1236	23.	1237	23.	1238	22.	1239	22.
1240	22.	1241	22.	1242	22.	1243	21.	1244	21.
1245	21.	1246	21.	1247	21.	1248	20.	1249	20.
1250	20.	1251	20.	1252	20.	1253	20.	1254	20.
1255	19.	1256	19.	1257	19.	1258	19.	1259	19.
1260	19.	1261	19.	1262	18.	1263	18.	1264	18.
1265	18.	1266	18.	1267	18.	1268	18.	1269	18.
1270	18.	1271	17.	1272	17.	1273	17.	1274	17.
1275	17.	1276	17.	1277	17.	1278	17.	1279	17.
1280	17.	1281	17.	1282	17.	1283	16.	1284	16.
1285	16.	1286	16.	1287	16.	1288	16.	1289	16.
1290	16.	1291	16.	1292	16.	1293	16.	1294	16.
1295	16.	1296	15.	1297	15.	1298	15.	1299	15.
1300	15.	1310	15.	1320	14.	1330	14.	1340	13.
1350	13.	1360	13.	1370	12.	1380	12.	1390	12.
1400	12.	1420	11.	1440	11.	1460	10.	1500	10.

Total Runoff = 37.567 Acre-Ft.
Peak Q = 160 CFS
Time to Peak Q = 1159 Minutes

APPENDIX 6

LACDPW MORA CALCULATIONS: PROPOSED CONDITION

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4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Proposed
5 50300   15A  HYDROGRAPH AT OUTLET BASIN AREAs A
6 50300    1A 020  529.911A292 1220 01600                     G1
6 50300    2A 020  5 6.4 6A292  880 02700
6 50300    3B  20  5 4.4 5A292 1410 05300
6 50300    4B 020  519.614A292  760 00200
6 50300    5B 020  5 6.9 7A294  132 04000
6 50300    6B 020  5 5.9 7A29
6 50300    7B  20 3013.3 9A294  917 00200
6 50300    8AB020         A29
6 50300    9C 020  5 7.7 8A294  540 00400
6 50300   10C  20  5 8.811A294  250 00100
6 50300   11C  20 8011.818A294 1610 00100
6 50300   12C  20  5 2.216A294  360 00100
6 50300   13C  20 80 7.522A294  630 00100
6 50300   14AC020         A29
6 50300   15A 020  0    50A29                             1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Proposed STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
50300	1A	30.	42.	30.	42.	2	1220.	0.01600	0.00	0.00	0.	20	11	A29	0.05
50300	2A	6.	12.	36.	38.	2	880.	0.02700	0.00	0.00	0.	20	6	A29	0.05
50300	3B	4.	9.	4.	9.	2	1410.	0.05300	0.00	0.00	0.	20	5	A29	0.05
50300	4B	20.	24.	24.	29.	2	760.	0.00200	0.00	0.00	0.	20	14	A29	0.05
50300	5B	7.	13.	31.	23.	4	132.	0.04000	2.00	0.00	0.	20	7	A29	0.05
50300	6B	6.	11.	37.	30.	0	0.	0.00000	0.00	0.00	0.	20	7	A29	0.05
50300	7B	13.	24.	50.	54.	4	917.	0.00200	3.75	0.00	0.	20	9	A29	0.30
50300	8AB	50.	50.	86.	82.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	9C	8.	13.	8.	13.	4	540.	0.00400	2.00	0.00	0.	20	8	A29	0.05
50300	10C	9.	12.	17.	25.	4	250.	0.00100	3.25	0.00	0.	20	11	A29	0.05
50300	11C	12.	20.	29.	45.	4	1610.	0.00100	4.00	0.00	0.	20	18	A29	0.80
50300	12C	2.	2.	31.	39.	4	360.	0.00100	3.75	0.00	0.	20	16	A29	0.05
50300	13C	7.	11.	38.	47.	4	630.	0.00100	4.00	0.00	0.	20	22	A29	0.80
50300	14AC	38.	46.	124.	109.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	15A	0.	0.	124.	109.	0	0.	0.00000	0.00	0.00	0.	20	50	A29	0.00

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 50300 15A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	5.	200	5.	300	5.	400	6.
500	6.	600	6.	700	7.	800	7.	900	8.
1000	10.	1050	12.	1100	16.	1110	17.	1120	20.
1130	25.	1131	26.	1132	26.	1133	27.	1134	27.
1135	28.	1136	29.	1137	30.	1138	30.	1139	31.
1140	32.	1141	33.	1142	35.	1143	36.	1144	37.
1145	39.	1146	40.	1147	42.	1148	44.	1149	46.
1150	49.	1151	52.	1152	57.	1153	64.	1154	73.
1155	82.	1156	91.	1157	98.	1158	104.	1159	108.
1160	109.	1161	109.	1162	108.	1163	106.	1164	105.
1165	104.	1166	103.	1167	102.	1168	101.	1169	100.
1170	99.	1171	97.	1172	95.	1173	93.	1174	90.
1175	87.	1176	83.	1177	79.	1178	74.	1179	69.
1180	65.	1181	60.	1182	56.	1183	52.	1184	49.
1185	45.	1186	42.	1187	40.	1188	37.	1189	35.
1190	33.	1191	31.	1192	29.	1193	28.	1194	27.
1195	25.	1196	24.	1197	23.	1198	22.	1199	21.
1200	21.	1201	20.	1202	19.	1203	19.	1204	18.
1205	17.	1206	17.	1207	16.	1208	16.	1209	16.
1210	15.	1211	15.	1212	14.	1213	14.	1214	14.
1215	14.	1216	13.	1217	13.	1218	13.	1219	13.
1220	12.	1221	12.	1222	12.	1223	12.	1224	12.
1225	11.	1226	11.	1227	11.	1228	11.	1229	11.
1230	11.	1231	11.	1232	10.	1233	10.	1234	10.
1235	10.	1236	10.	1237	10.	1238	10.	1239	10.
1240	10.	1241	10.	1242	10.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	9.	1249	9.
1250	9.	1251	9.	1252	9.	1253	9.	1254	9.
1255	9.	1256	9.	1257	9.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	8.
1265	8.	1266	8.	1267	8.	1268	8.	1269	8.
1270	8.	1271	8.	1272	8.	1273	8.	1274	8.
1275	8.	1276	8.	1277	8.	1278	8.	1279	8.
1280	8.	1281	8.	1282	8.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	7.	1289	7.
1290	7.	1291	7.	1292	7.	1293	7.	1294	7.
1295	7.	1296	7.	1297	7.	1298	7.	1299	7.
1300	7.	1310	7.	1320	7.	1330	6.	1340	6.
1350	6.	1360	6.	1370	6.	1380	6.	1390	6.
1400	5.	1420	5.	1440	5.	1460	5.	1500	5.

Total Runoff = 19.355 Acre-Ft.
Peak Q = 109 CFS
Time to Peak Q = 1160 Minutes

OFFSITE R -RVE 7A.DAT

```
4LADEPTH.RDT
5   300    1A  50 YR River Village - Newhall Ranch - OFFSITE RVE 1A-RVE 7A Proposed
5   300    8A  HYDROGRAPH AT OUTLET BASIN AREAs A
6   300    1A 097 0018.419A294  110 01000                    G1
6   300    2A 097 0038.922A292  915 00500
6   300    3B 097 0015.3 7A291 1930 04800
6   300    4B 097 0044.315A29
6   300    5AB097          A292  550 00180                  1
6   300    6A 097 0035.3 9A294  200 01300
6   300    7A 02010015.529A294 1500 00700
6   300    8A 020       99A29                             1  2
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11- 1-2007 SITE LICENSEE: PSOMAS AND ASSOCIATES PAGE 1

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - OFFSITE RVE 1A-RVE 7A Pro STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	1A	18.	22.	18.	22.	4	110.	0.01000	2.00	0.00	0.	97	19	A29	0.00
300	2A	39.	43.	57.	65.	2	915.	0.00500	0.00	0.00	0.	97	22	A29	0.00
300	3B	15.	32.	15.	32.	1	1930.	0.04800	0.00	0.00	0.	97	7	A29	0.00
300	4B	44.	61.	59.	73.	0	0.	0.00000	0.00	0.00	0.	97	15	A29	0.00

CONFLUENCE Q'S

300 5A TA 1166 QA 61. QAB 110. QB 48. 300 5B TB 1160 QB 73. QBA 127. QA 55.

300 5AB TAB 1162 QAB 130. QA 59. QB 72.

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	5AB	59.	73.	116.	130.	2	550.	0.00180	0.00	0.00	0.	97	0	A29	0.00
300	6A	35.	65.	151.	135.	4	200.	0.01300	3.75	0.00	0.	97	9	A29	0.00
300	7A	15.	23.	166.	156.	4	1500.	0.00700	4.50	0.00	0.	20	29	A29	1.00
300	8A	0.	0.	166.	152.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 300 8A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	4.	200	4.	300	5.	400	5.
500	5.	600	5.	700	6.	800	6.	900	7.
1000	9.	1050	11.	1100	20.	1110	23.	1120	29.
1130	37.	1131	38.	1132	39.	1133	41.	1134	42.
1135	43.	1136	44.	1137	45.	1138	46.	1139	48.
1140	49.	1141	51.	1142	53.	1143	55.	1144	57.
1145	59.	1146	61.	1147	64.	1148	67.	1149	70.
1150	74.	1151	79.	1152	86.	1153	95.	1154	105.
1155	116.	1156	126.	1157	134.	1158	140.	1159	146.
1160	150.	1161	152.	1162	152.	1163	151.	1164	149.
1165	148.	1166	148.	1167	149.	1168	150.	1169	151.
1170	152.	1171	151.	1172	149.	1173	145.	1174	141.
1175	135.	1176	130.	1177	124.	1178	119.	1179	114.
1180	109.	1181	103.	1182	98.	1183	92.	1184	87.
1185	82.	1186	77.	1187	72.	1188	68.	1189	64.
1190	60.	1191	56.	1192	53.	1193	50.	1194	47.
1195	44.	1196	42.	1197	40.	1198	38.	1199	36.
1200	34.	1201	32.	1202	31.	1203	29.	1204	28.
1205	27.	1206	25.	1207	24.	1208	23.	1209	22.
1210	21.	1211	21.	1212	20.	1213	19.	1214	19.
1215	18.	1216	17.	1217	17.	1218	16.	1219	16.
1220	15.	1221	15.	1222	15.	1223	14.	1224	14.
1225	13.	1226	13.	1227	13.	1228	12.	1229	12.
1230	12.	1231	12.	1232	11.	1233	11.	1234	11.
1235	11.	1236	10.	1237	10.	1238	10.	1239	10.
1240	10.	1241	10.	1242	9.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	9.	1249	9.
1250	8.	1251	8.	1252	8.	1253	8.	1254	8.
1255	8.	1256	8.	1257	8.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	7.	1264	7.
1265	7.	1266	7.	1267	7.	1268	7.	1269	7.
1270	7.	1271	7.	1272	7.	1273	7.	1274	7.
1275	7.	1276	7.	1277	7.	1278	7.	1279	7.
1280	7.	1281	7.	1282	7.	1283	7.	1284	7.
1285	7.	1286	6.	1287	6.	1288	6.	1289	6.
1290	6.	1291	6.	1292	6.	1293	6.	1294	6.
1295	6.	1296	6.	1297	6.	1298	6.	1299	6.
1300	6.	1310	6.	1320	6.	1330	6.	1340	5.
1350	5.	1360	5.	1370	5.	1380	5.	1390	5.
1400	5.	1420	5.	1440	4.	1460	4.	1500	4.

Total Runoff = 21.503 Acre-Ft.
Peak Q = 152 CFS
Time to Peak Q = 1161 Minutes

MORA INPUT SUMMARY TABLE - LANDMARK PROPOSED CONDITION

Line Number	Subarea	MORA Subarea Code	soil t					I Type	Length	Slope
Line 1	RVE-9B	1A	20	65	7.04	10	A29	4	75	0.008
	RVE-10B	2B	20	72	0.69	6	A29	4	260	0.026
		3AB					A29	4	40	0.008
		4A								
LINE 2	RVE -11C	1A	20	63.2	1.02	5	A29			
	RVE -12C	2A	20	65.9	0.63	5	A29	4	45	0.008
	RVE -13C	3A	20	66.0	0.87	5	A29			
	RVE -14C	4A	20	91.0	0.2	5	A29	4	755	0.008
	RVE -15C	5A	20	91.0	0.54	10	A29			
	RVE -16C	6A	20	64.2	2.78	10	A29	4	10	0.008
	RVE -17C	7A	20	68.8	5.17	8	A29	4	15	0.008
	RVE -18C	8A	20	91.0	0.66	11	A29			
	RVE -19C	9A	20	91.0	0.3	7	A29	4	220	0.008
	RVE-20C	10A	20	16.0	4.68	18	A29	4	50	0.05
		11A								
LINE 3	RVE-21D	1A	20	64.2	1.82	7	A29			
	RVE-22D	2A	20	91.0	0.34	7	A29	4	30	0.008
	RVE-22.1D	3A	20	74.0	3.56	7	A29	4	25	0.008
	RVE-23D	4A	20	91.0	0.55	10	A29			
	RVE-24D	5A	20	91.0	0.77	12	A29	4	180	0.008
	RVE-25D	6A	20	91.0	0.21	5	A29	4	35	0.008
	RVE-26D	7A	20	74.0	5.09	11	A29	4	45	0.008
	RVE-27D	8A	20	91.0	0.95	13	A29	4	10	0.01
	RVE-28D	9A	20	72.8	1.78	11	A29	4	185	0.02
	RVE-29D	10A	20	14.4	2.08	11	A29	4	760	0.01
		11A								
LINE 4	RVE-30E	1A	20	91.0	0.53	9	A29	4	275	0.005
	RVE-31E	2A	20	91.0	0.47	5	A29	4		
	RVE-32E	3A	20	91.0	0.33	5	A29	4	545	0.008
	RVE-33E	4B	20	70.7	5.86	9	A29	4	205	0.006
		5AB						4	200	0.006
	RVE-34E	6A	20	71.3	5.29	7	A29	4	75	0.006
	RVE-35E	7A	20	70.3	1.23	8	A29	4	50	0.01
	RVE-36E	8A	20	91.0	0.68	10	A29	4	190	0.02
	RVE-37E	9A	20	16.9	1.7	10	A29	4	60	0.05
		10A								
LINE 5	RVE-38F	1A	20	91.0	0.25	6	A29	4	60	0.009
	RVE-39F	2A	20	74.0	2.03	6	A29	4	37	0.009
	RVE-40F	3A	20	91.0	0.05	5	A29	4	565	0.009
	RVE-41F	4A	20	67.0	1.79	9	A29			
	RVE-42F	5A	20	91.0	0.5	9	A29	4	34	0.009
	RVE-43F	6A	20	67.7	3.86	12	A29	4	33	0.009
	RVE-44F	7A	20	66.4	2.86	11	A29	4	28	0.009
	RVE-45F	8A	20	91.0	0.14	5	A29	4	250	0.005
	RVE-46F	9A	20	17.4	2.69	13	A29	4	65	0.05
		10A	20							
LINE 6	RVE-51G	1A	20	67.3	2.99	8				
	RVE-52G	2A	20	66.3	1.24	7	A29	4	60	0.007
	RVE-53G	3A	20	65.4	1.21	6				
	RVE-54G	4A	20	65.5	1.19	6	A29	4	190	0.007
	RVE-55G	5A	20	64.4	0.82	5	A29	4	60	0.007
	RVE-56G	6A	20	69.0	1.02	7	A29	4	405	0.007
	RVE-57G	7A	20	67.8	3.37	9	A29	4	910	0.005
	RVE-58G/59G	8A	20	73.3	3.83	12	A29	4	15	0.005
	RVE-60G	9A	20	74.0	9.88	11	A29	4	70	0.005
	RVE-61G	10A	20	91.0	1.36	9				
	RVE-62G	11A	20	91.0	1.22	8	A29	4	870	0.006
	RVE-63G	12A	20	70.9	4.11	10	A29	4	1070	0.008
	RVE-8A	13B	20	15	6.35	8	A29	4	180	0.02
		14AB							370	0.03
		15A								
LINE 7	RVC-1A	1A	20	78.9	11.66	12	A29	4	400	0.007
	RVC-2A	2A	20	74	4.08	9	A29	4	30	0.007
	RVC-3A	3A	20	91	2.39	14				

Line Number	Subarea	MORA Subarea Code	soil type	impervious	area	Tc	Mass Curve	Channel Type	Length	Slope
	RVC-4A	4A	20	91	1.67	13	A29	4	110	0.007
	RVC-5A	5A	20	91	0.4	5				
	RVC-6A	6A	20	91	0.78	5	A29	4	35	0.007
	RVC-7A	7A	20	50	0.55	5	A29	4	810	0.006
	RVC-8A	8B	20	15.8	1.52	7	A29	4	450	0.01
		9AB							95	0.05
		10A								
LINE 8	RVE-15C	1A	20	15	3.54	8	A29	4	450	0.01
		2A							350	0.01
		3A							200	0.05
		4A								
LINE 9	RVC-9B	1A	20	74.0	8.39	11	A29	4	110	0.007
	RVC-10B	2A	20	91.0	1.03	9				
	RVC-11B	3A	20	91.0	0.74	8	A29	4	70	0.007
	RVC-12B	4A	20	82.0	9.79	10	A29	4	80	0.007
	RVC-13B	5A	20	91.0	1.34	11				
	RVC-14B	6A	20	71.0	2.83	11	A29	4	310	0.005
		7A							100	0.05
		8A								
LINE 10	RVW-1A	1A	20	91.0	0.8	6.0	A29	4	30	0.005
	RVW-2A	2A	20	74.0	3.2	5.0	A29	4	270	0.01
	RVW-3A	3A	20	69.1	1.5	7.0				
	RVW-4A	4A	20	63.7	3.3	8.0	A29	4	210	0.01
		5A	20						580	0.008
		6A	20						160	0.05
		7A	20							
LINE 11	RVW-5B	1A	20	91	0.34	5	A29	4	25	0.007
	RVW-6B	2A	20	74	1.21	5				
	RVW-7B	3A	20	91	0.18	5	A29	4	35	0.007
	RVW-8B	4A	20	91	0.2	5	A29	4	460	0.007
	RVW-9B	5A	20	70.8	2.22	7	A29	4	40	0.007
	RVW-10B	6A	20	76.5	1.33	7				
	RVW-11B	7A	20	65.8	0.34	5	A29	4	45	0.007
	RVW-12B	8A	20	16	3.54	15	A29	4	195	0.05
		9A								
LINE 12	RVW-12C	1A	20	91	0.87	12	A29	4	95	0.007
	RVW-13C	2A	20	65.4	3.6	8	A29	4	330	0.008
	RVW-14C	3A	20	68.2	2.58	12	A29	4	35	0.007
	RVW-15C	4A	20	65.3	2.13	9	A29	4	50	0.007
	RVW-16C	5A	20	91	0.99	7	A29	4	60	0.02
	RVW-17C	6A	20	15.6	1.91	11	A29	4	110	0.05
		7A								
LINE 13	RVW-18D	1A	20	74	1.93	7	A29	4	185	0.008
	RVW-19D	2A	20	91	0.51	6	A29	4	15	0.008
	RVW-20D	3A	20	91	0.38	6	A29	4	1065	0.008
	RVW-21D	4A	20	74	11	15	A29	4	120	0.008
	RVW-22D	5A	20	91	1.88	13	A29	4	25	0.008
	RVW-23D	6A	20	91	1.26	12	A29	4	22	0.008
	RVW-24D	7A	20	74	4.91	11	A29	4	275	0.008
	RVW-25D	8A	20	91	0.49	6	A29	4	30	0.008
	RVW-26.1D	9A	20	91	0.56	5	A29	4	275	0.008
	RVW-26D	10A	20	91	1.11	5	A29	4	40	0.008
	RVW-27D	11A	20	91	1.02	6	A29	4	100	0.008
	RVW-28D	12A	20	74	17.71	11				
	RVW-29D	13A	20	74	4.06	8	A29	4	810	0.008
	RVW-42E	14B	20	13.1	1.04	7	A29	4	180	0.01
		15AB					A29	4	125	0.05
		16A								
LINE 14	RVW-31E	1A	20	91	2.17	10	A29	4	50	0.005
	RVW-32E	2A	20	91	0.7	5	A29	4	50	0.005
	RVW-33E	3A	20	91	1.99	7	A29	4	275	0.005
	RVW-34E	4B	20	91	2.44	10	A29	4	275	0.012
		5AB								
		6A								
	RVW-35E	7C	20	91	0.65	7	A29	4	60	0.007

Line Number	Subarea	MORA Subarea Code	soil t					I Type	Length	Slope
	RVW-36E	8C	20	91	0.26	5	A29	4	80	0.007
		9AC							290	0.01
	RVW-37E	10A	20	91	0.37	5				
	RVW-38E	11A	20	91	0.77	6	A29	4	280	0.007
	RVW-40E	12A	20	91	7.41	10	A29	4	40	0.007
	RVW-39E	13D	20	78.517483	5.72	11	A29	4	500	0.005
	RVW-41E/39.1E	14D	20	50	3.1	10				
		15AD							100	0.05
		16A								
LINE-15	43G	1A	20	15.0	1.29	8	A29	4	500	0.007
	44G	2B	20	9.8	1.94	5	A29	4	50	0.01
		3AB							70	0.05
		4A								

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 1"
5 12134    4A  HYDROGRAPH AT OUTLET Line 1
6 12134    1A 020 6507.210A294   75 00800                    G1

6 12134    2B  20 7200.606A294  260 02600
6 12134    3AB020          A294   40 00800                  1

6 12134    4A 020  0    99A29                               1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 1" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	7.	15.	7.	15.	4	75.	0.00800	2.00	0.00	0.	20	10	A29	0.65
12134	2B	1.	3.	1.	3.	4	260.	0.02600	2.00	0.00	0.	20	6	A29	0.72

CONFLUENCE Q'S

12134 3A TA 1154 QA 15. QAB 18. QB 3. 12134 3B TB 1154 QB 3. QBA 18. QA 15.

12134 3AB TAB 1154 QAB 18. QA 15. QB 3.

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	3AB	1.	3.	8.	18.	4	40.	0.00800	2.00	0.00	0.	20	0	A29	0.00
12134	4A	0.	0.	8.	17.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET Line 1

HYDROGRAPH AT 12134 4A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	2.
1000	2.	1050	2.	1100	3.	1110	3.	1120	4.
1130	4.	1131	5.	1132	5.	1133	5.	1134	5.
1135	5.	1136	5.	1137	5.	1138	5.	1139	6.
1140	6.	1141	6.	1142	6.	1143	6.	1144	7.
1145	7.	1146	7.	1147	8.	1148	8.	1149	9.
1150	11.	1151	13.	1152	15.	1153	16.	1154	17.
1155	17.	1156	17.	1157	16.	1158	15.	1159	14.
1160	12.	1161	10.	1162	8.	1163	6.	1164	5.
1165	5.	1166	4.	1167	4.	1168	4.	1169	3.
1170	3.	1171	3.	1172	3.	1173	3.	1174	3.
1175	3.	1176	3.	1177	3.	1178	3.	1179	3.
1180	2.	1181	2.	1182	2.	1183	2.	1184	2.
1185	2.	1186	2.	1187	2.	1188	2.	1189	2.
1190	2.	1191	2.	1192	2.	1193	2.	1194	2.
1195	2.	1196	2.	1197	2.	1198	2.	1199	2.
1200	2.	1201	2.	1202	2.	1203	2.	1204	2.
1205	2.	1206	2.	1207	2.	1208	2.	1209	2.
1210	2.	1211	2.	1212	2.	1213	2.	1214	2.
1215	2.	1216	2.	1217	2.	1218	2.	1219	2.
1220	2.	1221	2.	1222	2.	1223	2.	1224	2.
1225	2.	1226	2.	1227	2.	1228	2.	1229	2.
1230	2.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	1.
1240	2.	1241	2.	1242	2.	1243	1.	1244	1.
1245	1.	1246	1.	1247	1.	1248	1.	1249	1.
1250	1.	1251	1.	1252	1.	1253	1.	1254	1.
1255	1.	1256	1.	1257	1.	1258	1.	1259	1.
1260	1.	1261	1.	1262	1.	1263	1.	1264	1.
1265	1.	1266	1.	1267	1.	1268	1.	1269	1.
1270	1.	1271	1.	1272	1.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 2.992 Acre-Ft.
Peak Q = 17 CFS
Time to Peak Q = 1154 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 2"
5 12134    11A  HYDROGRAPH AT OUTLET Line 2
6 12134    1A 020 6301.005A29                                    G1
6 12134    2A 020 6600.605A294   45 00800
6 12134    3A 020 6600.805A29
6 12134    4A 020 9100.205A294  755 00800
6 12134    5A 020 9100.510A29
6 12134    6A 020 6402.810A294   10 00800
6 12134    7A 020 6905.108A294   15 00800
6 12134    8A 020 9100.611A29
6 12134    9A 020 9100.307A294  220 00800
6 12134   10A 020 1604.718A294   50 05000
6 12134   11A 020       99A29                               1  2
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50 YR River Village - Newhall Ranch - Landmark "Line 2" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	1.	3.	1.	3.	0	0.	0.00000	0.00	0.00	0.	20	5	A29	0.63
12134	2A	1.	3.	2.	6.	4	45.	0.00800	2.00	0.00	0.	20	5	A29	0.66
12134	3A	1.	3.	3.	9.	0	0.	0.00000	0.00	0.00	0.	20	5	A29	0.66
12134	4A	0.	0.	3.	9.	4	755.	0.00800	2.00	0.00	0.	20	5	A29	0.91
12134	5A	0.	0.	3.	8.	0	0.	0.00000	0.00	0.00	0.	20	10	A29	0.91
12134	6A	3.	6.	6.	14.	4	10.	0.00800	2.00	0.00	0.	20	10	A29	0.64
12134	7A	5.	12.	11.	25.	4	15.	0.00800	2.25	0.00	0.	20	8	A29	0.69
12134	8A	1.	2.	12.	28.	0	0.	0.00000	0.00	0.00	0.	20	11	A29	0.91
12134	9A	0.	0.	12.	28.	4	220.	0.00800	2.25	0.00	0.	20	7	A29	0.91
12134	10A	5.	6.	17.	33.	4	50.	0.05000	2.00	0.00	0.	20	18	A29	0.16
12134	11A	0.	0.	17.	33.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT 12134 11A

HYDROGRAPH AT 12134 11A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	2.	200	2.	300	2.	400	2.
500	2.	600	2.	700	2.	800	2.	900	2.
1000	3.	1050	4.	1100	5.	1110	6.	1120	7.
1130	8.	1131	8.	1132	8.	1133	8.	1134	9.
1135	9.	1136	9.	1137	9.	1138	10.	1139	10.
1140	10.	1141	11.	1142	11.	1143	12.	1144	12.
1145	13.	1146	13.	1147	14.	1148	15.	1149	16.
1150	19.	1151	22.	1152	25.	1153	28.	1154	31.
1155	32.	1156	33.	1157	33.	1158	31.	1159	28.
1160	25.	1161	21.	1162	18.	1163	15.	1164	14.
1165	12.	1166	11.	1167	10.	1168	9.	1169	8.
1170	7.	1171	6.	1172	6.	1173	6.	1174	5.
1175	5.	1176	5.	1177	5.	1178	5.	1179	5.
1180	4.	1181	4.	1182	4.	1183	4.	1184	4.
1185	4.	1186	4.	1187	4.	1188	4.	1189	4.
1190	4.	1191	4.	1192	4.	1193	3.	1194	3.
1195	3.	1196	3.	1197	3.	1198	3.	1199	3.
1200	3.	1201	3.	1202	3.	1203	3.	1204	3.
1205	3.	1206	3.	1207	3.	1208	3.	1209	3.
1210	3.	1211	3.	1212	3.	1213	3.	1214	3.
1215	3.	1216	3.	1217	3.	1218	3.	1219	3.
1220	3.	1221	3.	1222	3.	1223	3.	1224	3.
1225	3.	1226	3.	1227	3.	1228	3.	1229	3.
1230	3.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	2.	1320	2.	1330	2.	1340	2.
1350	2.	1360	2.	1370	2.	1380	2.	1390	2.
1400	2.	1420	2.	1440	2.	1460	2.	1500	2.

Total Runoff = 5.428 Acre-Ft.
Peak Q = 33 CFS
Time to Peak Q = 1156 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 3"
5 12134   11A  HYDROGRAPH AT OUTLET Line 3
6 12134    1A 020 6401.807A29                                    G1
6 12134    2A 020 9100.307A294   30 00800
6 12134    3A 020 7403.507A29    25 00800
6 12134    4A 020 9100.510A29
6 12134    5A 020 9100.812A294  180 00800
6 12134    6A 020 9100.205A294   35 00800
6 12134    7A 020 7405.111A294   45 00800
6 12134    8A 020 9100.913A294   10 01000
6 12134    9A 020 7301.811A294  185 02000
6 12134   10A 020 1402.111A294  760 01000
6 12134   11A 020       99A29                               1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 3" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	2.	5.	2.	5.	0	0.	0.00000	0.00	0.00	0.	20	7	A29	0.64
12134	2A	0.	0.	2.	5.	4	30.	0.00800	2.00	0.00	0.	20	7	A29	0.91
12134	3A	3.	8.	5.	13.	4	25.	0.00800	2.00	0.00	0.	20	7	A29	0.74
12134	4A	0.	0.	5.	13.	0	0.	0.00000	0.00	0.00	0.	20	10	A29	0.91
12134	5A	1.	2.	6.	15.	4	180.	0.00800	2.00	0.00	0.	20	12	A29	0.91
12134	6A	0.	0.	6.	15.	4	35.	0.00800	2.00	0.00	0.	20	5	A29	0.91
12134	7A	5.	11.	11.	25.	4	45.	0.00800	2.25	0.00	0.	20	11	A29	0.74
12134	8A	1.	2.	12.	28.	4	10.	0.01000	2.25	0.00	0.	20	13	A29	0.91
12134	9A	2.	4.	14.	32.	4	185.	0.02000	2.00	0.00	0.	20	11	A29	0.73
12134	10A	2.	3.	16.	34.	4	760.	0.01000	2.50	0.00	0.	20	11	A29	0.14
12134	11A	0.	0.	16.	34.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 3

HYDROGRAPH AT 12134 11A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	2.	200	2.	300	2.	400	2.
500	2.	600	2.	700	2.	800	2.	900	3.
1000	3.	1050	4.	1100	5.	1110	6.	1120	7.
1130	8.	1131	9.	1132	9.	1133	9.	1134	9.
1135	9.	1136	9.	1137	10.	1138	10.	1139	10.
1140	10.	1141	11.	1142	11.	1143	11.	1144	12.
1145	12.	1146	13.	1147	14.	1148	14.	1149	15.
1150	16.	1151	18.	1152	20.	1153	24.	1154	27.
1155	30.	1156	32.	1157	34.	1158	34.	1159	33.
1160	31.	1161	29.	1162	26.	1163	23.	1164	19.
1165	16.	1166	14.	1167	12.	1168	10.	1169	9.
1170	8.	1171	8.	1172	7.	1173	7.	1174	7.
1175	6.	1176	6.	1177	6.	1178	6.	1179	6.
1180	5.	1181	5.	1182	5.	1183	5.	1184	5.
1185	5.	1186	5.	1187	5.	1188	5.	1189	4.
1190	4.	1191	4.	1192	4.	1193	4.	1194	4.
1195	4.	1196	4.	1197	4.	1198	4.	1199	4.
1200	4.	1201	4.	1202	4.	1203	4.	1204	4.
1205	4.	1206	4.	1207	3.	1208	3.	1209	3.
1210	3.	1211	3.	1212	3.	1213	3.	1214	3.
1215	3.	1216	3.	1217	3.	1218	3.	1219	3.
1220	3.	1221	3.	1222	3.	1223	3.	1224	3.
1225	3.	1226	3.	1227	3.	1228	3.	1229	3.
1230	3.	1231	3.	1232	3.	1233	3.	1234	3.
1235	3.	1236	3.	1237	3.	1238	3.	1239	3.
1240	3.	1241	3.	1242	3.	1243	3.	1244	3.
1245	3.	1246	3.	1247	3.	1248	3.	1249	3.
1250	3.	1251	3.	1252	3.	1253	3.	1254	3.
1255	3.	1256	3.	1257	3.	1258	3.	1259	3.
1260	3.	1261	3.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	2.	1320	2.	1330	2.	1340	2.
1350	2.	1360	2.	1370	2.	1380	2.	1390	2.
1400	2.	1420	2.	1440	2.	1460	2.	1500	2.

Total Runoff = 5.701 Acre-Ft.
Peak Q = 34 CFS
Time to Peak Q = 1157 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 4"
5 12134   10A  HYDROGRAPH AT OUTLET Line 4
6 12134    1A 020 9100.509A294  275 00500                    G1
6 12134    2A 020 9100.505A29
6 12134    3A 020 9100.305A294  545 00800
6 12134    4B 020 7105.809A294  205 00600
6 12134    5AB020         A294  200 00600
6 12134    6A 020 7105.307A294   75 00600
6 12134    7A 020 7001.208A294   50 01000
6 12134    8A 020 9100.710A294  190 02000
6 12134    9A 020 1701.710A294   60 05000
6 12134   10A 020       99A29                            1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 4" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	0.	0.	0.	0.	4	275.	0.00500	0.00	0.00	0.	20	9	A29	0.91
12134	2A	0.	0.	0.	0.	0	0.	0.00000	0.00	0.00	0.	20	5	A29	0.91
12134	3A	0.	0.	0.	0.	4	545.	0.00800	0.00	0.00	0.	20	5	A29	0.91
12134	4B	6.	14.	6.	14.	4	205.	0.00600	2.00	0.00	0.	20	9	A29	0.71
12134	5AB	6.	14.	6.	14.	4	200.	0.00600	2.00	0.00	0.	20	0	A29	0.00
12134	6A	5.	13.	11.	26.	4	75.	0.00600	2.50	0.00	0.	20	7	A29	0.71
12134	7A	1.	2.	12.	28.	4	50.	0.01000	2.25	0.00	0.	20	8	A29	0.70
12134	8A	1.	2.	13.	30.	4	190.	0.02000	2.00	0.00	0.	20	10	A29	0.91
12134	9A	2.	3.	15.	33.	4	60.	0.05000	2.00	0.00	0.	20	10	A29	0.17
12134	10A	0.	0.	15.	33.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 4

HYDROGRAPH AT 12134 10A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	2.
500	2.	600	2.	700	2.	800	2.	900	2.
1000	3.	1050	4.	1100	5.	1110	6.	1120	7.
1130	8.	1131	8.	1132	8.	1133	9.	1134	9.
1135	9.	1136	9.	1137	9.	1138	10.	1139	10.
1140	10.	1141	11.	1142	11.	1143	12.	1144	12.
1145	13.	1146	13.	1147	14.	1148	15.	1149	16.
1150	19.	1151	22.	1152	25.	1153	29.	1154	32.
1155	33.	1156	33.	1157	32.	1158	30.	1159	27.
1160	23.	1161	19.	1162	16.	1163	13.	1164	11.
1165	10.	1166	9.	1167	8.	1168	7.	1169	7.
1170	6.	1171	6.	1172	6.	1173	6.	1174	5.
1175	5.	1176	5.	1177	5.	1178	5.	1179	5.
1180	5.	1181	4.	1182	4.	1183	4.	1184	4.
1185	4.	1186	4.	1187	4.	1188	4.	1189	4.
1190	4.	1191	4.	1192	4.	1193	4.	1194	4.
1195	4.	1196	3.	1197	3.	1198	3.	1199	3.
1200	3.	1201	3.	1202	3.	1203	3.	1204	3.
1205	3.	1206	3.	1207	3.	1208	3.	1209	3.
1210	3.	1211	3.	1212	3.	1213	3.	1214	3.
1215	3.	1216	3.	1217	3.	1218	3.	1219	3.
1220	3.	1221	3.	1222	3.	1223	3.	1224	3.
1225	3.	1226	3.	1227	3.	1228	3.	1229	3.
1230	3.	1231	3.	1232	3.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	2.	1320	2.	1330	2.	1340	2.
1350	2.	1360	2.	1370	2.	1380	2.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 4.837 Acre-Ft.
Peak Q = 33 CFS
Time to Peak Q = 1155 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 5"
5 12134   10A  HYDROGRAPH AT OUTLET Line 5
6 12134    1A 020 9100.306A294   60 00900                    G1
6 12134    2A 020 7402.006A294   37 00900
6 12134    3A 020 9100.105A294  565 00900
6 12134    4A 020 6701.809A29
6 12134    5A 020 9100.509A294   34 00900
6 12134    6A 020 6703.912A294   33 00900
6 12134    7A 020 6602.911A294   28 00900
6 12134    8A 020 9100.105A294  250 00500
6 12134    9A 020 1702.713A294   65 05000
6 12134   10A 020       99A29                          1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 5" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	0.	0.	0.	0.	4	60.	0.00900	0.00	0.00	0.	20	6	A29	0.91
12134	2A	2.	6.	2.	6.	4	37.	0.00900	2.00	0.00	0.	20	6	A29	0.74
12134	3A	0.	0.	2.	6.	4	565.	0.00900	2.00	0.00	0.	20	5	A29	0.91
12134	4A	2.	4.	4.	10.	0	0.	0.00000	0.00	0.00	0.	20	9	A29	0.67
12134	5A	0.	0.	4.	10.	4	34.	0.00900	2.00	0.00	0.	20	9	A29	0.91
12134	6A	4.	8.	8.	17.	4	33.	0.00900	2.00	0.00	0.	20	12	A29	0.67
12134	7A	3.	6.	11.	23.	4	28.	0.00900	2.25	0.00	0.	20	11	A29	0.66
12134	8A	0.	0.	11.	23.	4	250.	0.00500	2.25	0.00	0.	20	5	A29	0.91
12134	9A	3.	4.	14.	27.	4	65.	0.05000	2.00	0.00	0.	20	13	A29	0.17
12134	10A	0.	0.	14.	27.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 5

HYDROGRAPH AT 12134 10A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	2.	600	2.	700	2.	800	2.	900	2.
1000	2.	1050	3.	1100	4.	1110	5.	1120	6.
1130	7.	1131	7.	1132	7.	1133	7.	1134	7.
1135	8.	1136	8.	1137	8.	1138	8.	1139	8.
1140	9.	1141	9.	1142	9.	1143	10.	1144	10.
1145	11.	1146	11.	1147	12.	1148	12.	1149	13.
1150	15.	1151	17.	1152	20.	1153	23.	1154	25.
1155	26.	1156	27.	1157	27.	1158	27.	1159	26.
1160	24.	1161	22.	1162	19.	1163	16.	1164	13.
1165	11.	1166	9.	1167	7.	1168	7.	1169	6.
1170	6.	1171	5.	1172	5.	1173	5.	1174	5.
1175	5.	1176	4.	1177	4.	1178	4.	1179	4.
1180	4.	1181	4.	1182	4.	1183	4.	1184	4.
1185	4.	1186	3.	1187	3.	1188	3.	1189	3.
1190	3.	1191	3.	1192	3.	1193	3.	1194	3.
1195	3.	1196	3.	1197	3.	1198	3.	1199	3.
1200	3.	1201	3.	1202	3.	1203	3.	1204	3.
1205	3.	1206	3.	1207	3.	1208	3.	1209	3.
1210	2.	1211	2.	1212	2.	1213	2.	1214	2.
1215	2.	1216	2.	1217	2.	1218	2.	1219	2.
1220	2.	1221	2.	1222	2.	1223	2.	1224	2.
1225	2.	1226	2.	1227	2.	1228	2.	1229	2.
1230	2.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	2.	1320	2.	1330	2.	1340	2.
1350	2.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 4.254 Acre-Ft.
Peak Q = 27 CFS
Time to Peak Q = 1156 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 6"
5 12134   15A  HYDROGRAPH AT OUTLET Line 6
6 12134    1A 020 6703.008A29                                    G1
6 12134    2A 020 6601.207A294   60 00700
6 12134    3A 020 6501.206A29
6 12134    4A 020 6601.206A294  190 00700
6 12134    5A 020 6500.805A294   60 00700
6 12134    6A 020 6901.007A294  405 00700
6 12134    7A 020 6803.409A294  910 00500
6 12134    8A 020 7303.812A294   15 00500
6 12134    9A 020 7409.911A294   70 00500
6 12134   10A 020 9101.409A29
6 12134   11A 020 9101.208A294  870 00600
6 12134   12A 020 7104.110A294 1070 00800
6 12134   13B 020 1606.408A294  180 02000
6 12134   14AB020         A294  370 03000
6 12134   15A 020       99A29                                1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 6" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	3.	7.	3.	7.	0	0.	0.00000	0.00	0.00	0.	20	8	A29	0.67
12134	2A	1.	3.	4.	10.	4	60.	0.00700	2.00	0.00	0.	20	7	A29	0.66
12134	3A	1.	3.	5.	12.	0	0.	0.00000	0.00	0.00	0.	20	6	A29	0.65
12134	4A	1.	3.	6.	15.	4	190.	0.00700	2.00	0.00	0.	20	6	A29	0.66
12134	5A	1.	3.	7.	17.	4	60.	0.00700	2.00	0.00	0.	20	5	A29	0.65
12134	6A	1.	3.	8.	20.	4	405.	0.00700	2.00	0.00	0.	20	7	A29	0.69
12134	7A	3.	7.	11.	26.	4	910.	0.00500	2.50	0.00	0.	20	9	A29	0.68
12134	8A	4.	8.	15.	32.	4	15.	0.00500	2.75	0.00	0.	20	12	A29	0.73
12134	9A	10.	21.	25.	50.	4	70.	0.00500	3.00	0.00	0.	20	11	A29	0.74
12134	10A	1.	3.	26.	52.	0	0.	0.00000	0.00	0.00	0.	20	9	A29	0.91
12134	11A	1.	3.	27.	54.	4	870.	0.00600	3.00	0.00	0.	20	8	A29	0.91
12134	12A	4.	9.	31.	58.	4	1070.	0.00800	3.00	0.00	0.	20	10	A29	0.71
12134	13B	6.	11.	6.	11.	4	180.	0.02000	2.00	0.00	0.	20	8	A29	0.16
12134	14AB	6.	11.	37.	59.	4	370.	0.03000	2.50	0.00	0.	20	0	A29	0.00
12134	15A	0.	0.	37.	59.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 6

HYDROGRAPH AT 12134 15A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	4.	300	4.	400	4.
500	4.	600	4.	700	5.	800	5.	900	6.
1000	7.	1050	9.	1100	11.	1110	12.	1120	15.
1130	17.	1131	18.	1132	18.	1133	18.	1134	19.
1135	19.	1136	19.	1137	20.	1138	20.	1139	21.
1140	21.	1141	22.	1142	22.	1143	23.	1144	23.
1145	24.	1146	25.	1147	26.	1148	27.	1149	29.
1150	31.	1151	33.	1152	36.	1153	40.	1154	43.
1155	47.	1156	51.	1157	54.	1158	56.	1159	58.
1160	59.	1161	59.	1162	59.	1163	59.	1164	59.
1165	58.	1166	55.	1167	52.	1168	47.	1169	42.
1170	37.	1171	33.	1172	29.	1173	26.	1174	24.
1175	21.	1176	20.	1177	18.	1178	17.	1179	16.
1180	15.	1181	15.	1182	14.	1183	13.	1184	13.
1185	13.	1186	12.	1187	12.	1188	11.	1189	11.
1190	11.	1191	11.	1192	10.	1193	10.	1194	10.
1195	10.	1196	10.	1197	10.	1198	9.	1199	9.
1200	9.	1201	9.	1202	9.	1203	9.	1204	9.
1205	8.	1206	8.	1207	8.	1208	8.	1209	8.
1210	8.	1211	8.	1212	8.	1213	8.	1214	8.
1215	8.	1216	7.	1217	7.	1218	7.	1219	7.
1220	7.	1221	7.	1222	7.	1223	7.	1224	7.
1225	7.	1226	7.	1227	7.	1228	7.	1229	7.
1230	7.	1231	7.	1232	7.	1233	6.	1234	6.
1235	6.	1236	6.	1237	6.	1238	6.	1239	6.
1240	6.	1241	6.	1242	6.	1243	6.	1244	6.
1245	6.	1246	6.	1247	6.	1248	6.	1249	6.
1250	6.	1251	6.	1252	6.	1253	6.	1254	6.
1255	6.	1256	6.	1257	6.	1258	6.	1259	6.
1260	6.	1261	6.	1262	6.	1263	5.	1264	5.
1265	5.	1266	5.	1267	5.	1268	5.	1269	5.
1270	5.	1271	5.	1272	5.	1273	5.	1274	5.
1275	5.	1276	5.	1277	5.	1278	5.	1279	5.
1280	5.	1281	5.	1282	5.	1283	5.	1284	5.
1285	5.	1286	5.	1287	5.	1288	5.	1289	5.
1290	5.	1291	5.	1292	5.	1293	5.	1294	5.
1295	5.	1296	5.	1297	5.	1298	5.	1299	5.
1300	5.	1310	5.	1320	4.	1330	4.	1340	4.
1350	4.	1360	4.	1370	4.	1380	4.	1390	4.
1400	4.	1420	4.	1440	4.	1460	3.	1500	3.

Total Runoff = 12.169 Acre-Ft.
Peak Q = 59 CFS
Time to Peak Q = 1160 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 7"
5 12134   10A  HYDROGRAPH AT OUTLET Line 7
6 12134    1A 020 7911.712A294  400 00700                    G1
6 12134    2A 020 7404.109A294   30 00700
6 12134    3A 020 9102.414A29
6 12134    4A 020 9101.713A294  110 00700
6 12134    5A 020 9100.405A29    60 00700
6 12134    6A 020 9100.805A294   35 00700
6 12134    7A 020 5000.605A294  810 00600
6 12134    8B 020 1601.507A294  450 01000
6 12134    9AB020         A294   95 05000
6 12134   10A 020       99A29                            1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 7" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	12.	25.	12.	25.	4	400.	0.00700	2.25	0.00	0.	20	12	A29	0.79
12134	2A	4.	9.	16.	34.	4	30.	0.00700	2.50	0.00	0.	20	9	A29	0.74
12134	3A	2.	4.	18.	38.	0	0.	0.00000	0.00	0.00	0.	20	14	A29	0.91
12134	4A	2.	4.	20.	42.	4	110.	0.00700	2.75	0.00	0.	20	13	A29	0.91
12134	5A	0.	0.	20.	42.	4	60.	0.00700	2.75	0.00	0.	20	5	A29	0.91
12134	6A	1.	3.	21.	44.	4	35.	0.00700	2.75	0.00	0.	20	5	A29	0.91
12134	7A	1.	3.	22.	45.	4	810.	0.00600	3.00	0.00	0.	20	5	A29	0.50
12134	8B	1.	2.	1.	2.	4	450.	0.01000	2.00	0.00	0.	20	7	A29	0.16
12134	9AB	1.	2.	23.	46.	4	95.	0.05000	2.00	0.00	0.	20	0	A29	0.00
12134	10A	0.	0.	23.	46.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 7

HYDROGRAPH AT 12134 10A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	3.	400	3.
500	3.	600	4.	700	4.	800	4.	900	5.
1000	6.	1050	7.	1100	9.	1110	10.	1120	11.
1130	13.	1131	13.	1132	13.	1133	14.	1134	14.
1135	14.	1136	15.	1137	15.	1138	15.	1139	15.
1140	16.	1141	16.	1142	17.	1143	17.	1144	18.
1145	18.	1146	19.	1147	20.	1148	21.	1149	22.
1150	23.	1151	25.	1152	28.	1153	31.	1154	35.
1155	40.	1156	43.	1157	45.	1158	46.	1159	46.
1160	45.	1161	43.	1162	41.	1163	39.	1164	36.
1165	33.	1166	30.	1167	26.	1168	23.	1169	20.
1170	18.	1171	16.	1172	14.	1173	13.	1174	12.
1175	12.	1176	11.	1177	11.	1178	10.	1179	10.
1180	10.	1181	9.	1182	9.	1183	9.	1184	9.
1185	8.	1186	8.	1187	8.	1188	8.	1189	8.
1190	8.	1191	7.	1192	7.	1193	7.	1194	7.
1195	7.	1196	7.	1197	7.	1198	7.	1199	7.
1200	7.	1201	6.	1202	6.	1203	6.	1204	6.
1205	6.	1206	6.	1207	6.	1208	6.	1209	6.
1210	6.	1211	6.	1212	6.	1213	6.	1214	6.
1215	6.	1216	6.	1217	6.	1218	5.	1219	5.
1220	5.	1221	5.	1222	5.	1223	5.	1224	5.
1225	5.	1226	5.	1227	5.	1228	5.	1229	5.
1230	5.	1231	5.	1232	5.	1233	5.	1234	5.
1235	5.	1236	5.	1237	5.	1238	5.	1239	5.
1240	5.	1241	5.	1242	5.	1243	5.	1244	5.
1245	5.	1246	5.	1247	5.	1248	5.	1249	5.
1250	4.	1251	4.	1252	4.	1253	4.	1254	4.
1255	4.	1256	4.	1257	4.	1258	4.	1259	4.
1260	4.	1261	4.	1262	4.	1263	4.	1264	4.
1265	4.	1266	4.	1267	4.	1268	4.	1269	4.
1270	4.	1271	4.	1272	4.	1273	4.	1274	4.
1275	4.	1276	4.	1277	4.	1278	4.	1279	4.
1280	4.	1281	4.	1282	4.	1283	4.	1284	4.
1285	4.	1286	4.	1287	4.	1288	4.	1289	4.
1290	4.	1291	4.	1292	4.	1293	4.	1294	4.
1295	4.	1296	4.	1297	4.	1298	4.	1299	4.
1300	4.	1310	4.	1320	4.	1330	3.	1340	3.
1350	3.	1360	3.	1370	3.	1380	3.	1390	3.
1400	3.	1420	3.	1440	3.	1460	3.	1500	3.

Total Runoff = 9.561 Acre-Ft.
Peak Q = 46 CFS
Time to Peak Q = 1158 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 8"
5 12134    4A  HYDROGRAPH AT OUTLET Line 8
6 12134    1A 020 1503.508A294  450 01000                    G1
6 12134    2A 020       99A29
6 12134    3A 020       99A29
6 12134    4A 020       99A29                              1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 8"

STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	3.	5.	3.	5.	4	450.	0.01000	2.00	0.00	0.	20	8	A29	0.15
12134	2A	0.	0.	3.	5.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
12134	3A	0.	0.	3.	5.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
12134	4A	0.	0.	3.	5.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 8

HYDROGRAPH AT 12134 4A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	0.	200	0.	300	0.	400	0.
500	0.	600	0.	700	0.	800	0.	900	0.
1000	0.	1050	0.	1100	0.	1110	0.	1120	1.
1130	1.	1131	1.	1132	1.	1133	1.	1134	1.
1135	1.	1136	1.	1137	1.	1138	1.	1139	1.
1140	1.	1141	1.	1142	1.	1143	1.	1144	1.
1145	1.	1146	2.	1147	2.	1148	2.	1149	2.
1150	2.	1151	3.	1152	3.	1153	4.	1154	5.
1155	5.	1156	5.	1157	5.	1158	5.	1159	4.
1160	4.	1161	3.	1162	2.	1163	2.	1164	1.
1165	1.	1166	1.	1167	1.	1168	1.	1169	1.
1170	1.	1171	1.	1172	1.	1173	0.	1174	0.
1175	0.	1176	0.	1177	0.	1178	0.	1179	0.
1180	0.	1181	0.	1182	0.	1183	0.	1184	0.
1185	0.	1186	0.	1187	0.	1188	0.	1189	0.
1190	0.	1191	0.	1192	0.	1193	0.	1194	0.
1195	0.	1196	0.	1197	0.	1198	0.	1199	0.
1200	0.	1201	0.	1202	0.	1203	0.	1204	0.
1205	0.	1206	0.	1207	0.	1208	0.	1209	0.
1210	0.	1211	0.	1212	0.	1213	0.	1214	0.
1215	0.	1216	0.	1217	0.	1218	0.	1219	0.
1220	0.	1221	0.	1222	0.	1223	0.	1224	0.
1225	0.	1226	0.	1227	0.	1228	0.	1229	0.
1230	0.	1231	0.	1232	0.	1233	0.	1234	0.
1235	0.	1236	0.	1237	0.	1238	0.	1239	0.
1240	0.	1241	0.	1242	0.	1243	0.	1244	0.
1245	0.	1246	0.	1247	0.	1248	0.	1249	0.
1250	0.	1251	0.	1252	0.	1253	0.	1254	0.
1255	0.	1256	0.	1257	0.	1258	0.	1259	0.
1260	0.	1261	0.	1262	0.	1263	0.	1264	0.
1265	0.	1266	0.	1267	0.	1268	0.	1269	0.
1270	0.	1271	0.	1272	0.	1273	0.	1274	0.
1275	0.	1276	0.	1277	0.	1278	0.	1279	0.
1280	0.	1281	0.	1282	0.	1283	0.	1284	0.
1285	0.	1286	0.	1287	0.	1288	0.	1289	0.
1290	0.	1291	0.	1292	0.	1293	0.	1294	0.
1295	0.	1296	0.	1297	0.	1298	0.	1299	0.
1300	0.	1310	0.	1320	0.	1330	0.	1340	0.
1350	0.	1360	0.	1370	0.	1380	0.	1390	0.
1400	0.	1420	0.	1440	0.	1460	0.	1500	0.

Total Runoff = 0.137 Acre-Ft.
Peak Q = 5 CFS
Time to Peak Q = 1154 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 9"
5 12134   10A  HYDROGRAPH AT OUTLET Line 9
6 12134    1A 020 7408.411A294  110 00700                    G1
6 12134    2A 020 9101.009A29
6 12134    3A 020 9100.708A294   70 00700
6 12134    4A 020 8209.810A294   80 00700
6 12134    5A 020 9101.311A29
6 12134    6A 020 7102.811A294  310 00500
6 12134    7A 020       99A294  100 05000
6 12134    8A 020       99A29                             1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 9" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	8.	17.	8.	17.	4	110.	0.00700	2.00	0.00	0.	20	11	A29	0.74
12134	2A	1.	3.	9.	19.	0	0.	0.00000	0.00	0.00	0.	20	9	A29	0.91
12134	3A	1.	3.	10.	22.	4	70.	0.00700	2.25	0.00	0.	20	8	A29	0.91
12134	4A	10.	23.	20.	45.	4	80.	0.00700	2.75	0.00	0.	20	10	A29	0.82
12134	5A	1.	2.	21.	47.	0	0.	0.00000	0.00	0.00	0.	20	11	A29	0.91
12134	6A	3.	6.	24.	53.	4	310.	0.00500	3.25	0.00	0.	20	11	A29	0.71
12134	7A	0.	0.	24.	53.	4	100.	0.05000	2.00	0.00	0.	20	99	A29	0.00
12134	8A	0.	0.	24.	53.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT 12134 8A

HYDROGRAPH AT 12134 8A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	3.	400	3.
500	3.	600	3.	700	4.	800	4.	900	5.
1000	6.	1050	7.	1100	9.	1110	11.	1120	13.
1130	15.	1131	15.	1132	15.	1133	15.	1134	16.
1135	16.	1136	16.	1137	17.	1138	17.	1139	18.
1140	18.	1141	19.	1142	19.	1143	20.	1144	21.
1145	22.	1146	23.	1147	24.	1148	25.	1149	27.
1150	30.	1151	34.	1152	39.	1153	44.	1154	49.
1155	52.	1156	53.	1157	53.	1158	51.	1159	50.
1160	46.	1161	42.	1162	36.	1163	31.	1164	25.
1165	21.	1166	18.	1167	16.	1168	14.	1169	13.
1170	12.	1171	12.	1172	11.	1173	11.	1174	10.
1175	10.	1176	10.	1177	10.	1178	9.	1179	9.
1180	9.	1181	9.	1182	8.	1183	8.	1184	8.
1185	8.	1186	8.	1187	8.	1188	7.	1189	7.
1190	7.	1191	7.	1192	7.	1193	7.	1194	7.
1195	7.	1196	7.	1197	7.	1198	6.	1199	6.
1200	6.	1201	6.	1202	6.	1203	6.	1204	6.
1205	6.	1206	6.	1207	6.	1208	6.	1209	6.
1210	6.	1211	6.	1212	6.	1213	6.	1214	5.
1215	5.	1216	5.	1217	5.	1218	5.	1219	5.
1220	5.	1221	5.	1222	5.	1223	5.	1224	5.
1225	5.	1226	5.	1227	5.	1228	5.	1229	5.
1230	5.	1231	5.	1232	5.	1233	5.	1234	5.
1235	5.	1236	5.	1237	5.	1238	5.	1239	5.
1240	5.	1241	5.	1242	5.	1243	5.	1244	4.
1245	4.	1246	4.	1247	4.	1248	4.	1249	4.
1250	4.	1251	4.	1252	4.	1253	4.	1254	4.
1255	4.	1256	4.	1257	4.	1258	4.	1259	4.
1260	4.	1261	4.	1262	4.	1263	4.	1264	4.
1265	4.	1266	4.	1267	4.	1268	4.	1269	4.
1270	4.	1271	4.	1272	4.	1273	4.	1274	4.
1275	4.	1276	4.	1277	4.	1278	4.	1279	4.
1280	4.	1281	4.	1282	4.	1283	4.	1284	4.
1285	4.	1286	4.	1287	4.	1288	4.	1289	4.
1290	4.	1291	4.	1292	4.	1293	4.	1294	4.
1295	4.	1296	4.	1297	4.	1298	4.	1299	4.
1300	4.	1310	3.	1320	3.	1330	3.	1340	3.
1350	3.	1360	3.	1370	3.	1380	3.	1390	3.
1400	3.	1420	3.	1440	3.	1460	3.	1500	3.

Total Runoff = 9.490 Acre-Ft.
Peak Q = 53 CFS
Time to Peak Q = 1156 Minutes

```
4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 10"
5 12134    7A  HYDROGRAPH AT OUTLET Line 10
6 12134    1A 020 9100.806A294   30 00500                    G1
6 12134    2A 020 7403.205A294  270 01000
6 12134    3A 020 6901.507A29
6 12134    4A 020 6403.308A294  210 01000
6 12134    5A 020       99A294  580 00800
6 12134    6A 020       99A294  160 05000
6 12134    7A 020       99A29                              1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 10" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	1.	3.	1.	3.	4	30.	0.00500	2.00	0.00	0.	20	6	A29	0.91
12134	2A	3.	9.	4.	12.	4	270.	0.01000	2.00	0.00	0.	20	5	A29	0.74
12134	3A	1.	3.	5.	14.	0	0.	0.00000	0.00	0.00	0.	20	7	A29	0.69
12134	4A	3.	7.	8.	21.	4	210.	0.01000	2.00	0.00	0.	20	8	A29	0.64
12134	5A	0.	0.	8.	21.	4	580.	0.00800	2.00	0.00	0.	20	99	A29	0.00
12134	6A	0.	0.	8.	20.	4	160.	0.05000	2.00	0.00	0.	20	99	A29	0.00
12134	7A	0.	0.	8.	20.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 10

HYDROGRAPH AT 12134 7A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	2.
1000	2.	1050	2.	1100	3.	1110	3.	1120	4.
1130	4.	1131	4.	1132	5.	1133	5.	1134	5.
1135	5.	1136	5.	1137	5.	1138	5.	1139	5.
1140	6.	1141	6.	1142	6.	1143	6.	1144	6.
1145	7.	1146	7.	1147	7.	1148	8.	1149	8.
1150	9.	1151	10.	1152	11.	1153	13.	1154	15.
1155	17.	1156	19.	1157	20.	1158	20.	1159	18.
1160	16.	1161	13.	1162	11.	1163	9.	1164	7.
1165	6.	1166	5.	1167	5.	1168	5.	1169	4.
1170	4.	1171	4.	1172	4.	1173	3.	1174	3.
1175	3.	1176	3.	1177	3.	1178	3.	1179	3.
1180	3.	1181	3.	1182	3.	1183	3.	1184	3.
1185	2.	1186	2.	1187	2.	1188	2.	1189	2.
1190	2.	1191	2.	1192	2.	1193	2.	1194	2.
1195	2.	1196	2.	1197	2.	1198	2.	1199	2.
1200	2.	1201	2.	1202	2.	1203	2.	1204	2.
1205	2.	1206	2.	1207	2.	1208	2.	1209	2.
1210	2.	1211	2.	1212	2.	1213	2.	1214	2.
1215	2.	1216	2.	1217	2.	1218	2.	1219	2.
1220	2.	1221	2.	1222	2.	1223	2.	1224	2.
1225	2.	1226	2.	1227	2.	1228	2.	1229	2.
1230	2.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	1.
1255	1.	1256	1.	1257	1.	1258	1.	1259	1.
1260	1.	1261	1.	1262	1.	1263	1.	1264	1.
1265	1.	1266	1.	1267	1.	1268	1.	1269	1.
1270	1.	1271	1.	1272	1.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 3.042 Acre-Ft.
Peak Q = 20 CFS
Time to Peak Q = 1157 Minutes

```
4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 11"
5 12134    9A  HYDROGRAPH AT OUTLET Line 11
6 12134    1A 020 9100.305A294   25 00700                    G1
6 12134    2A 020 7401.205A29
6 12134    3A 020 9100.205A294   35 00700
6 12134    4A 020 9100.205A294  460 00700
6 12134    5A 020 7102.207A294   40 00700
6 12134    6A 020 7601.307A29
6 12134    7A 020 6600.305A294   45 00700
6 12134    8A 020 1603.515A294  195 05000
6 12134    9A 020       99A29                            1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 11" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	0.	0.	0.	0.	4	25.	0.00700	0.00	0.00	0.	20	5	A29	0.91
12134	2A	1.	3.	1.	3.	0	0.	0.00000	0.00	0.00	0.	20	5	A29	0.74
12134	3A	0.	0.	1.	3.	4	35.	0.00700	2.00	0.00	0.	20	5	A29	0.91
12134	4A	0.	0.	1.	3.	4	460.	0.00700	2.00	0.00	0.	20	5	A29	0.91
12134	5A	2.	5.	3.	8.	4	40.	0.00700	2.00	0.00	0.	20	7	A29	0.71
12134	6A	1.	3.	4.	10.	0	0.	0.00000	0.00	0.00	0.	20	7	A29	0.76
12134	7A	0.	0.	4.	10.	4	45.	0.00700	2.00	0.00	0.	20	5	A29	0.66
12134	8A	3.	4.	7.	14.	4	195.	0.05000	2.00	0.00	0.	20	15	A29	0.16
12134	9A	0.	0.	7.	14.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 11

HYDROGRAPH AT 12134 9A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	1.
1000	1.	1050	2.	1100	2.	1110	2.	1120	3.
1130	3.	1131	3.	1132	3.	1133	3.	1134	3.
1135	4.	1136	4.	1137	4.	1138	4.	1139	4.
1140	4.	1141	4.	1142	4.	1143	5.	1144	5.
1145	5.	1146	5.	1147	6.	1148	6.	1149	7.
1150	8.	1151	10.	1152	11.	1153	13.	1154	14.
1155	14.	1156	14.	1157	13.	1158	11.	1159	10.
1160	8.	1161	7.	1162	6.	1163	6.	1164	5.
1165	5.	1166	4.	1167	4.	1168	3.	1169	3.
1170	2.	1171	2.	1172	2.	1173	2.	1174	2.
1175	2.	1176	2.	1177	2.	1178	2.	1179	2.
1180	2.	1181	2.	1182	2.	1183	2.	1184	2.
1185	2.	1186	2.	1187	2.	1188	2.	1189	2.
1190	1.	1191	1.	1192	1.	1193	1.	1194	1.
1195	1.	1196	1.	1197	1.	1198	1.	1199	1.
1200	1.	1201	1.	1202	1.	1203	1.	1204	1.
1205	1.	1206	1.	1207	1.	1208	1.	1209	1.
1210	1.	1211	1.	1212	1.	1213	1.	1214	1.
1215	1.	1216	1.	1217	1.	1218	1.	1219	1.
1220	1.	1221	1.	1222	1.	1223	1.	1224	1.
1225	1.	1226	1.	1227	1.	1228	1.	1229	1.
1230	1.	1231	1.	1232	1.	1233	1.	1234	1.
1235	1.	1236	1.	1237	1.	1238	1.	1239	1.
1240	1.	1241	1.	1242	1.	1243	1.	1244	1.
1245	1.	1246	1.	1247	1.	1248	1.	1249	1.
1250	1.	1251	1.	1252	1.	1253	1.	1254	1.
1255	1.	1256	1.	1257	1.	1258	1.	1259	1.
1260	1.	1261	1.	1262	1.	1263	1.	1264	1.
1265	1.	1266	1.	1267	1.	1268	1.	1269	1.
1270	1.	1271	1.	1272	1.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 2.485 Acre-Ft.
Peak Q = 14 CFS
Time to Peak Q = 1154 Minutes

```
4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 12"
5 12134    7A  HYDROGRAPH AT OUTLET Line 12
6 12134    1A 020 9100.912A294   95 00700                    G1
6 12134    2A 020 6503.008A294  330 00800
6 12134    3A 020 6802.012A294   35 00700
6 12134    4A 020 6502.109A294   50 00700
6 12134    5A 020 9101.007A294   60 02000
6 12134    6A 020 1601.911A294  110 05000
6 12134    7A 020       99A29                              1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 12" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	1.	2.	1.	2.	4	95.	0.00700	2.00	0.00	0.	20	12	A29	0.91
12134	2A	3.	7.	4.	9.	4	330.	0.00800	2.00	0.00	0.	20	8	A29	0.65
12134	3A	2.	4.	6.	13.	4	35.	0.00700	2.00	0.00	0.	20	12	A29	0.68
12134	4A	2.	4.	8.	17.	4	50.	0.00700	2.00	0.00	0.	20	9	A29	0.65
12134	5A	1.	3.	9.	20.	4	60.	0.02000	2.00	0.00	0.	20	7	A29	0.91
12134	6A	2.	3.	11.	23.	4	110.	0.05000	2.00	0.00	0.	20	11	A29	0.16
12134	7A	0.	0.	11.	23.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET Line 12

HYDROGRAPH AT 12134 7A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	2.	800	2.	900	2.
1000	2.	1050	3.	1100	3.	1110	4.	1120	5.
1130	6.	1131	6.	1132	6.	1133	6.	1134	6.
1135	6.	1136	6.	1137	7.	1138	7.	1139	7.
1140	7.	1141	8.	1142	8.	1143	8.	1144	9.
1145	9.	1146	9.	1147	10.	1148	11.	1149	12.
1150	14.	1151	16.	1152	18.	1153	21.	1154	22.
1155	23.	1156	23.	1157	22.	1158	21.	1159	19.
1160	17.	1161	14.	1162	12.	1163	10.	1164	8.
1165	7.	1166	6.	1167	6.	1168	5.	1169	5.
1170	5.	1171	4.	1172	4.	1173	4.	1174	4.
1175	4.	1176	4.	1177	3.	1178	3.	1179	3.
1180	3.	1181	3.	1182	3.	1183	3.	1184	3.
1185	3.	1186	3.	1187	3.	1188	3.	1189	3.
1190	3.	1191	3.	1192	3.	1193	3.	1194	3.
1195	3.	1196	2.	1197	2.	1198	2.	1199	2.
1200	2.	1201	2.	1202	2.	1203	2.	1204	2.
1205	2.	1206	2.	1207	2.	1208	2.	1209	2.
1210	2.	1211	2.	1212	2.	1213	2.	1214	2.
1215	2.	1216	2.	1217	2.	1218	2.	1219	2.
1220	2.	1221	2.	1222	2.	1223	2.	1224	2.
1225	2.	1226	2.	1227	2.	1228	2.	1229	2.
1230	2.	1231	2.	1232	2.	1233	2.	1234	2.
1235	2.	1236	2.	1237	2.	1238	2.	1239	2.
1240	2.	1241	2.	1242	2.	1243	2.	1244	2.
1245	2.	1246	2.	1247	2.	1248	2.	1249	2.
1250	2.	1251	2.	1252	2.	1253	2.	1254	2.
1255	2.	1256	2.	1257	2.	1258	2.	1259	2.
1260	2.	1261	2.	1262	2.	1263	2.	1264	2.
1265	2.	1266	2.	1267	2.	1268	2.	1269	2.
1270	2.	1271	2.	1272	2.	1273	2.	1274	2.
1275	2.	1276	2.	1277	2.	1278	2.	1279	2.
1280	2.	1281	2.	1282	2.	1283	2.	1284	2.
1285	2.	1286	2.	1287	2.	1288	2.	1289	2.
1290	2.	1291	2.	1292	2.	1293	2.	1294	2.
1295	2.	1296	2.	1297	2.	1298	2.	1299	2.
1300	2.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 3.650 Acre-Ft.
Peak Q = 23 CFS
Time to Peak Q = 1155 Minutes

```
4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 13"
5 12134   16A  HYDROGRAPH AT OUTLET Line 13
6 12134    1A 020 7401.907A294  185 00800                    G1
6 12134    2A 020 9100.506A294   15 00800
6 12134    3A 020 9100.306A294 1065 00800
6 12134    4A 020 7411.015A294  120 00800
6 12134    5A 020 9101.813A294   25 00800
6 12134    6A 020 9101.312A294   22 00800
6 12134    7A 020 7404.911A294  275 00800
6 12134    8A 020 9100.506A294   30 00800
6 12134    9A 020 9100.505A294  275 00800
6 12134   10A 020 9101.105A294   40 00800
6 12134   11A 020 9101.006A294  100 00800
6 12134   12A 020 7417.711A29
6 12134   13A 020 7404.008A294  810 00800
6 12134   14B 020 1301.007A294  180 01000
6 12134   15AB020         A294  125 05000
6 12134   16A 020       99A29                             1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 13" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	2.	5.	2.	5.	4	185.	0.00800	2.00	0.00	0.	20	7	A29	0.74
12134	2A	0.	0.	2.	5.	4	15.	0.00800	2.00	0.00	0.	20	6	A29	0.91
12134	3A	0.	0.	2.	5.	4	1065.	0.00800	2.00	0.00	0.	20	6	A29	0.91
12134	4A	11.	20.	13.	24.	4	120.	0.00800	2.25	0.00	0.	20	15	A29	0.74
12134	5A	2.	4.	15.	28.	4	25.	0.00800	2.25	0.00	0.	20	13	A29	0.91
12134	6A	1.	2.	16.	30.	4	22.	0.00800	2.50	0.00	0.	20	12	A29	0.91
12134	7A	5.	11.	21.	40.	4	275.	0.00800	2.75	0.00	0.	20	11	A29	0.74
12134	8A	0.	0.	21.	40.	4	30.	0.00800	2.75	0.00	0.	20	6	A29	0.91
12134	9A	0.	0.	21.	40.	4	275.	0.00800	2.75	0.00	0.	20	5	A29	0.91
12134	10A	1.	3.	22.	41.	4	40.	0.00800	2.75	0.00	0.	20	5	A29	0.91
12134	11A	1.	3.	23.	43.	4	100.	0.00800	2.75	0.00	0.	20	6	A29	0.91
12134	12A	18.	38.	41.	80.	0	0.	0.00000	0.00	0.00	0.	20	11	A29	0.74
12134	13A	4.	10.	45.	89.	4	810.	0.00800	3.50	0.00	0.	20	8	A29	0.74
12134	14B	1.	2.	1.	2.	4	180.	0.01000	2.00	0.00	0.	20	7	A29	0.13
12134	15AB	1.	2.	46.	89.	4	125.	0.05000	2.50	0.00	0.	20	0	A29	0.00
12134	16A	0.	0.	46.	89.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 13

HYDROGRAPH AT 12134 16A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	5.	200	5.	300	6.	400	6.
500	6.	600	7.	700	7.	800	8.	900	9.
1000	11.	1050	13.	1100	17.	1110	19.	1120	22.
1130	26.	1131	26.	1132	27.	1133	27.	1134	28.
1135	28.	1136	29.	1137	29.	1138	30.	1139	31.
1140	31.	1141	32.	1142	33.	1143	34.	1144	35.
1145	37.	1146	38.	1147	40.	1148	42.	1149	44.
1150	47.	1151	52.	1152	59.	1153	66.	1154	74.
1155	81.	1156	86.	1157	88.	1158	89.	1159	88.
1160	86.	1161	82.	1162	77.	1163	71.	1164	65.
1165	58.	1166	52.	1167	47.	1168	42.	1169	38.
1170	35.	1171	32.	1172	29.	1173	26.	1174	24.
1175	23.	1176	21.	1177	20.	1178	20.	1179	19.
1180	18.	1181	18.	1182	17.	1183	17.	1184	16.
1185	16.	1186	16.	1187	15.	1188	15.	1189	15.
1190	14.	1191	14.	1192	14.	1193	14.	1194	13.
1195	13.	1196	13.	1197	13.	1198	13.	1199	13.
1200	12.	1201	12.	1202	12.	1203	12.	1204	12.
1205	12.	1206	12.	1207	11.	1208	11.	1209	11.
1210	11.	1211	11.	1212	11.	1213	11.	1214	11.
1215	11.	1216	10.	1217	10.	1218	10.	1219	10.
1220	10.	1221	10.	1222	10.	1223	10.	1224	10.
1225	10.	1226	10.	1227	10.	1228	10.	1229	9.
1230	9.	1231	9.	1232	9.	1233	9.	1234	9.
1235	9.	1236	9.	1237	9.	1238	9.	1239	9.
1240	9.	1241	9.	1242	9.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	8.	1249	8.
1250	8.	1251	8.	1252	8.	1253	8.	1254	8.
1255	8.	1256	8.	1257	8.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	8.
1265	8.	1266	8.	1267	8.	1268	8.	1269	8.
1270	8.	1271	8.	1272	8.	1273	8.	1274	8.
1275	8.	1276	8.	1277	8.	1278	7.	1279	7.
1280	7.	1281	7.	1282	7.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	7.	1289	7.
1290	7.	1291	7.	1292	7.	1293	7.	1294	7.
1295	7.	1296	7.	1297	7.	1298	7.	1299	7.
1300	7.	1310	7.	1320	7.	1330	6.	1340	6.
1350	6.	1360	6.	1370	6.	1380	6.	1390	6.
1400	6.	1420	5.	1440	5.	1460	5.	1500	5.

Total Runoff = 17.864 Acre-Ft.
Peak Q = 89 CFS
Time to Peak Q = 1158 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 14"
5 12134   16A  HYDROGRAPH AT OUTLET Line 14
6 12134    1A 020 9102.210A294   50 00500                    G1
6 12134    2A 020 9100.705A294   50 00500
6 12134    3A 020 9102.007A294  275 00500
6 12134    4A 020 9102.410A294  275 01200
6 12134    5AB020         A29
6 12134    6A 020       99A29
6 12134    7C 020 9100.607A294   60 00700
6 12134    8C 020 9100.305A294   80 00700
6 12134    9AC020         A29
6 12134   10A 020 9100.305A29
6 12134   11A 020 9100.706A294  280 00700
6 12134   12A 020 9107.410A294   40 00700
6 12134   13D 020 7805.711A294  500 00500
6 12134   14D 020 5003.110A29
6 12134   15AD020         A294  100 05000
6 12134   16A 020       99A29                            1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 14" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	2.	5.	2.	5.	4	50.	0.00500	2.00	0.00	0.	20	10	A29	0.91
12134	2A	1.	3.	3.	8.	4	50.	0.00500	2.00	0.00	0.	20	5	A29	0.91
12134	3A	2.	6.	5.	13.	4	275.	0.00500	2.00	0.00	0.	20	7	A29	0.91
12134	4A	2.	5.	7.	18.	4	275.	0.01200	2.00	0.00	0.	20	10	A29	0.91
12134	5AB	0.	0.	7.	18.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
12134	6A	0.	0.	7.	18.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00
12134	7C	1.	3.	1.	3.	4	60.	0.00700	2.00	0.00	0.	20	7	A29	0.91
12134	8C	0.	0.	1.	3.	4	80.	0.00700	2.00	0.00	0.	20	5	A29	0.91
12134	9AC	1.	3.	8.	20.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
12134	10A	0.	0.	8.	20.	0	0.	0.00000	0.00	0.00	0.	20	5	A29	0.91
12134	11A	1.	3.	9.	23.	4	280.	0.00700	2.25	0.00	0.	20	6	A29	0.91
12134	12A	7.	17.	16.	39.	4	40.	0.00700	2.75	0.00	0.	20	10	A29	0.91
12134	13D	6.	13.	6.	13.	4	500.	0.00500	2.00	0.00	0.	20	11	A29	0.78
12134	14D	3.	6.	9.	18.	0	0.	0.00000	0.00	0.00	0.	20	10	A29	0.50
12134	15AD	9.	18.	25.	57.	4	100.	0.05000	2.25	0.00	0.	20	0	A29	0.00
12134	16A	0.	0.	25.	57.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET Line 14

HYDROGRAPH AT 12134 16A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	3.	400	4.
500	4.	600	4.	700	4.	800	5.	900	5.
1000	6.	1050	8.	1100	10.	1110	12.	1120	14.
1130	16.	1131	16.	1132	16.	1133	17.	1134	17.
1135	17.	1136	18.	1137	18.	1138	19.	1139	19.
1140	20.	1141	20.	1142	21.	1143	22.	1144	22.
1145	23.	1146	25.	1147	26.	1148	27.	1149	30.
1150	34.	1151	38.	1152	43.	1153	49.	1154	53.
1155	56.	1156	57.	1157	57.	1158	55.	1159	51.
1160	46.	1161	41.	1162	35.	1163	29.	1164	25.
1165	22.	1166	19.	1167	17.	1168	16.	1169	14.
1170	14.	1171	13.	1172	12.	1173	12.	1174	11.
1175	11.	1176	11.	1177	10.	1178	10.	1179	10.
1180	10.	1181	9.	1182	9.	1183	9.	1184	9.
1185	9.	1186	8.	1187	8.	1188	8.	1189	8.
1190	8.	1191	8.	1192	8.	1193	8.	1194	7.
1195	7.	1196	7.	1197	7.	1198	7.	1199	7.
1200	7.	1201	7.	1202	7.	1203	7.	1204	7.
1205	7.	1206	7.	1207	6.	1208	6.	1209	6.
1210	6.	1211	6.	1212	6.	1213	6.	1214	6.
1215	6.	1216	6.	1217	6.	1218	6.	1219	6.
1220	6.	1221	6.	1222	6.	1223	6.	1224	6.
1225	6.	1226	6.	1227	6.	1228	6.	1229	5.
1230	5.	1231	5.	1232	5.	1233	5.	1234	5.
1235	5.	1236	5.	1237	5.	1238	5.	1239	5.
1240	5.	1241	5.	1242	5.	1243	5.	1244	5.
1245	5.	1246	5.	1247	5.	1248	5.	1249	5.
1250	5.	1251	5.	1252	5.	1253	5.	1254	5.
1255	5.	1256	5.	1257	5.	1258	5.	1259	5.
1260	5.	1261	5.	1262	5.	1263	5.	1264	5.
1265	5.	1266	5.	1267	5.	1268	4.	1269	5.
1270	4.	1271	4.	1272	4.	1273	4.	1274	4.
1275	4.	1276	4.	1277	4.	1278	4.	1279	4.
1280	4.	1281	4.	1282	4.	1283	4.	1284	4.
1285	4.	1286	4.	1287	4.	1288	4.	1289	4.
1290	4.	1291	4.	1292	4.	1293	4.	1294	4.
1295	4.	1296	4.	1297	4.	1298	4.	1299	4.
1300	4.	1310	4.	1320	4.	1330	4.	1340	4.
1350	4.	1360	4.	1370	4.	1380	3.	1390	3.
1400	3.	1420	3.	1440	3.	1460	3.	1500	3.

Total Runoff = 10.475 Acre-Ft.
Peak Q = 57 CFS
Time to Peak Q = 1156 Minutes

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4LADEPTH.RDT
5 12134    1A  50 YR River Village - Newhall Ranch - Landmark  "Line 15"
5 12134    4A  HYDROGRAPH AT OUTLET Line 2
6 12134    1A 020 1501.308A294  500 00700                    G1
6 12134    2B 020 1001.905A294   50 01000
6 12134    3AB020          A294   70 05000
6 12134    4A 020       99A29                              1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Landmark "Line 15" STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
12134	1A	1.	2.	1.	2.	4	500.	0.00700	2.00	0.00	0.	20	8	A29	0.15
12134	2B	2.	5.	2.	5.	4	50.	0.01000	2.00	0.00	0.	20	5	A29	0.10
12134	3AB	2.	5.	3.	6.	4	70.	0.05000	2.00	0.00	0.	20	0	A29	0.00
12134	4A	0.	0.	3.	5.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

HYDROGRAPH AT OUTLET Line 15

HYDROGRAPH AT 12134 4A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	1.	200	1.	300	1.	400	1.
500	1.	600	1.	700	1.	800	1.	900	1.
1000	1.	1050	1.	1100	1.	1110	1.	1120	1.
1130	1.	1131	1.	1132	1.	1133	1.	1134	1.
1135	1.	1136	1.	1137	1.	1138	1.	1139	1.
1140	1.	1141	1.	1142	1.	1143	1.	1144	1.
1145	2.	1146	2.	1147	2.	1148	2.	1149	2.
1150	3.	1151	4.	1152	5.	1153	5.	1154	5.
1155	5.	1156	4.	1157	4.	1158	3.	1159	2.
1160	2.	1161	2.	1162	2.	1163	1.	1164	1.
1165	1.	1166	1.	1167	1.	1168	1.	1169	1.
1170	1.	1171	1.	1172	1.	1173	1.	1174	1.
1175	1.	1176	1.	1177	1.	1178	1.	1179	1.
1180	1.	1181	1.	1182	1.	1183	1.	1184	1.
1185	1.	1186	1.	1187	1.	1188	1.	1189	1.
1190	1.	1191	1.	1192	1.	1193	1.	1194	1.
1195	1.	1196	1.	1197	1.	1198	1.	1199	1.
1200	1.	1201	1.	1202	1.	1203	1.	1204	1.
1205	1.	1206	1.	1207	1.	1208	1.	1209	1.
1210	1.	1211	1.	1212	1.	1213	1.	1214	1.
1215	1.	1216	1.	1217	1.	1218	1.	1219	1.
1220	1.	1221	1.	1222	1.	1223	1.	1224	1.
1225	1.	1226	1.	1227	1.	1228	1.	1229	1.
1230	1.	1231	1.	1232	1.	1233	1.	1234	1.
1235	1.	1236	1.	1237	1.	1238	1.	1239	1.
1240	1.	1241	1.	1242	1.	1243	1.	1244	1.
1245	1.	1246	1.	1247	1.	1248	1.	1249	1.
1250	1.	1251	1.	1252	1.	1253	1.	1254	1.
1255	1.	1256	1.	1257	1.	1258	1.	1259	1.
1260	1.	1261	1.	1262	1.	1263	1.	1264	1.
1265	1.	1266	1.	1267	1.	1268	1.	1269	1.
1270	1.	1271	1.	1272	1.	1273	1.	1274	1.
1275	1.	1276	1.	1277	1.	1278	1.	1279	1.
1280	1.	1281	1.	1282	1.	1283	1.	1284	1.
1285	1.	1286	1.	1287	1.	1288	1.	1289	1.
1290	1.	1291	1.	1292	1.	1293	1.	1294	1.
1295	1.	1296	1.	1297	1.	1298	1.	1299	1.
1300	1.	1310	1.	1320	1.	1330	1.	1340	1.
1350	1.	1360	1.	1370	1.	1380	1.	1390	1.
1400	1.	1420	1.	1440	1.	1460	1.	1500	1.

Total Runoff = 2.050 Acre-Ft.
Peak Q = 5 CFS
Time to Peak Q = 1152 Minutes

APPENDIX 7

LACDPW MORA CALCULATIONS: PROPOSED CONDITION - BURNED

```
 4LADEPTH.RDT
 5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Proposed BURNED
 5 50300   15A  HYDROGRAPH AT OUTLET BASIN AREAs A
 6 50300    1A 198  538.011A292 1220 01600                    G1
 6 50300    2A 198  5 8.0 6A292  880 02700
 6 50300    3B 198  5 4.4 5A292 1410 05300
 6 50300    4B 198  519.614A292  760 00200
 6 50300    5B 198  5 6.9 7A294  132 04000
 6 50300    6B 198  5 5.9 7A29
 6 50300    7B 198 3013.3 9A294  917 00200
 6 50300    8AB198         A29
 6 50300    9C 198  5 7.7 8A294  540 00400
 6 50300   10C 198  5 8.811A294  250 00100
 6 50300   11C  20 8011.818A294 1610 00100
 6 50300   12C  20  5 2.216A294  360 00100
 6 50300   13C  20 80 7.522A294  630 00100
 6 50300   14AC020         A29
 6 50300   15A 020  0    50A29                             1  2

**NOTE**
        THE AREA FOR SUBAREAS 1A AND 2A HAVE BEEN INCREASED TO ACCOUNT FOR BURNED AND BULKED FLOWRATE.
        AREA 1A ACTUAL AREA IS 30.0 ACRES
        AREA 2A ACTUAL AREA IS 6.0 ACRES
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11-11-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Proposed BURNED

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
50300	1A	38.	68.	38.	68.	2	1220.	0.01600	0.00	0.00	0.	198	11	A29	0.05
50300	2A	8.	20.	46.	67.	2	880.	0.02700	0.00	0.00	0.	198	6	A29	0.05
50300	3B	4.	11.	4.	11.	2	1410.	0.05300	0.00	0.00	0.	198	5	A29	0.05
50300	4B	20.	31.	24.	38.	2	760.	0.00200	0.00	0.00	0.	198	14	A29	0.05
50300	5B	7.	16.	31.	34.	4	132.	0.04000	2.00	0.00	0.	198	7	A29	0.05
50300	6B	6.	14.	37.	42.	0	0.	0.00000	0.00	0.00	0.	198	7	A29	0.05
50300	7B	13.	28.	50.	70.	4	917.	0.00200	4.25	0.00	0.	198	9	A29	0.30
50300	8AB	50.	65.	96.	127.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	9C	8.	17.	8.	17.	4	540.	0.00400	2.25	0.00	0.	198	8	A29	0.05
50300	10C	9.	16.	17.	32.	4	250.	0.00100	3.50	0.00	0.	198	11	A29	0.05
50300	11C	12.	20.	29.	52.	4	1610.	0.00100	4.25	0.00	0.	20	18	A29	0.80
50300	12C	2.	2.	31.	46.	4	360.	0.00100	4.00	0.00	0.	20	16	A29	0.05
50300	13C	7.	11.	38.	54.	4	630.	0.00100	4.25	0.00	0.	20	22	A29	0.80
50300	14AC	38.	52.	134.	159.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	15A	0.	0.	134.	159.	0	0.	0.00000	0.00	0.00	0.	20	50	A29	0.00

NOTE

THE AREA FOR SUBAREAS 1A AND 2A HAVE BEEN INCREASED TO ACCOUNT FOR BURNED AND BULKED FLOWRATE.
AREA 1A ACTUAL AREA IS 30.0 ACRES
AREA 2A ACTUAL AREA IS 6.0 ACRES

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 50300 15A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	10.	200	11.	300	11.	400	12.
500	12.	600	13.	700	14.	800	16.	900	18.
1000	22.	1050	26.	1100	33.	1110	36.	1120	41.
1130	47.	1131	48.	1132	48.	1133	49.	1134	50.
1135	51.	1136	52.	1137	53.	1138	54.	1139	55.
1140	56.	1141	57.	1142	59.	1143	60.	1144	62.
1145	64.	1146	66.	1147	68.	1148	71.	1149	74.
1150	77.	1151	82.	1152	90.	1153	100.	1154	112.
1155	124.	1156	136.	1157	146.	1158	153.	1159	157.
1160	159.	1161	158.	1162	156.	1163	153.	1164	150.
1165	148.	1166	146.	1167	144.	1168	141.	1169	138.
1170	134.	1171	131.	1172	127.	1173	123.	1174	118.
1175	113.	1176	107.	1177	101.	1178	95.	1179	89.
1180	84.	1181	79.	1182	74.	1183	70.	1184	66.
1185	62.	1186	59.	1187	56.	1188	53.	1189	51.
1190	49.	1191	47.	1192	45.	1193	43.	1194	42.
1195	40.	1196	39.	1197	38.	1198	37.	1199	36.
1200	35.	1201	34.	1202	33.	1203	33.	1204	32.
1205	31.	1206	31.	1207	30.	1208	29.	1209	29.
1210	28.	1211	28.	1212	28.	1213	27.	1214	27.
1215	26.	1216	26.	1217	26.	1218	25.	1219	25.
1220	25.	1221	24.	1222	24.	1223	24.	1224	24.
1225	23.	1226	23.	1227	23.	1228	23.	1229	22.
1230	22.	1231	22.	1232	22.	1233	22.	1234	21.
1235	21.	1236	21.	1237	21.	1238	21.	1239	21.
1240	20.	1241	20.	1242	20.	1243	20.	1244	20.
1245	20.	1246	20.	1247	19.	1248	19.	1249	19.
1250	19.	1251	19.	1252	19.	1253	19.	1254	19.
1255	18.	1256	18.	1257	18.	1258	18.	1259	18.
1260	18.	1261	18.	1262	18.	1263	18.	1264	18.
1265	17.	1266	17.	1267	17.	1268	17.	1269	17.
1270	17.	1271	17.	1272	17.	1273	17.	1274	17.
1275	17.	1276	17.	1277	16.	1278	16.	1279	16.
1280	16.	1281	16.	1282	16.	1283	16.	1284	16.
1285	16.	1286	16.	1287	16.	1288	16.	1289	16.
1290	15.	1291	15.	1292	15.	1293	15.	1294	15.
1295	15.	1296	15.	1297	15.	1298	15.	1299	15.
1300	15.	1310	14.	1320	14.	1330	13.	1340	13.
1350	13.	1360	12.	1370	12.	1380	12.	1390	12.
1400	11.	1420	11.	1440	11.	1460	10.	1500	10.

Total Runoff = 37.255 Acre-Ft.
Peak Q = 159 CFS
Time to Peak Q = 1160 Minutes

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4LADEPTH.RDT
5   300    1A  50 YR River Village - Newhall Ranch - OFFSITE RVE 1A-RVE 7A BURNED
5   300    8A  HYDROGRAPH AT OUTLET BASIN AREAs A
6   300    1A 199 0018.419A294  110 01000                    G1
6   300    2A 199 0038.922A292  915 00500
6   300    3B 199 0015.3 7A291 1930 04800
6   300    4B 199 0044.315A29
6   300    5AB199         A292  550 00180                   1
6   300    6A 199 0035.3 9A294  200 01300
6   300    7A  2010015.529A294 1500 00700
6   300    8A  20       99A29                              1  2
```

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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - OFFSITE RVE 1A-RVE 7A BURNED STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	1A	18.	26.	18.	26.	4	110.	0.01000	2.25	0.00	0.	199	19	A29	0.00
300	2A	39.	52.	57.	79.	2	915.	0.00500	0.00	0.00	0.	199	22	A29	0.00
300	3B	15.	37.	15.	37.	1	1930.	0.04800	0.00	0.00	0.	199	7	A29	0.00
300	4B	44.	73.	59.	89.	0	0.	0.00000	0.00	0.00	0.	199	15	A29	0.00

CONFLUENCE Q'S

300 5A TA 1165 QA 75. QAB 144. QB 69.

300 5AB TAB 1162 QAB 160. QA 73.

300 5B TB 1160 QB 89. QBA 158. QA 69.

QB 87.

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	5AB	59.	89.	116.	160.	2	550.	0.00180	0.00	0.00	0.	199	0	A29	0.00
300	6A	35.	76.	151.	171.	4	200.	0.01300	4.00	0.00	0.	199	9	A29	0.00
300	7A	15.	23.	166.	192.	4	1500.	0.00700	4.75	0.00	0.	20	29	A29	1.00
300	8A	0.	0.	166.	188.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 300 8A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	7.	200	7.	300	8.	400	9.
500	9.	600	11.	700	12.	800	14.	900	16.
1000	20.	1050	25.	1100	35.	1110	39.	1120	46.
1130	56.	1131	57.	1132	58.	1133	60.	1134	61.
1135	62.	1136	64.	1137	65.	1138	67.	1139	69.
1140	70.	1141	72.	1142	75.	1143	77.	1144	79.
1145	82.	1146	85.	1147	88.	1148	91.	1149	95.
1150	99.	1151	105.	1152	114.	1153	124.	1154	135.
1155	147.	1156	157.	1157	166.	1158	173.	1159	180.
1160	184.	1161	187.	1162	188.	1163	188.	1164	186.
1165	184.	1166	184.	1167	185.	1168	186.	1169	187.
1170	186.	1171	185.	1172	181.	1173	176.	1174	170.
1175	163.	1176	156.	1177	149.	1178	142.	1179	136.
1180	130.	1181	124.	1182	118.	1183	111.	1184	105.
1185	99.	1186	93.	1187	88.	1188	83.	1189	79.
1190	74.	1191	70.	1192	66.	1193	63.	1194	60.
1195	57.	1196	54.	1197	52.	1198	49.	1199	47.
1200	45.	1201	44.	1202	42.	1203	40.	1204	39.
1205	38.	1206	37.	1207	35.	1208	34.	1209	33.
1210	33.	1211	32.	1212	31.	1213	30.	1214	30.
1215	29.	1216	28.	1217	28.	1218	27.	1219	27.
1220	26.	1221	26.	1222	25.	1223	25.	1224	25.
1225	24.	1226	24.	1227	23.	1228	23.	1229	23.
1230	22.	1231	22.	1232	22.	1233	22.	1234	21.
1235	21.	1236	21.	1237	21.	1238	20.	1239	20.
1240	20.	1241	20.	1242	20.	1243	19.	1244	19.
1245	19.	1246	19.	1247	19.	1248	19.	1249	18.
1250	18.	1251	18.	1252	18.	1253	18.	1254	18.
1255	17.	1256	17.	1257	17.	1258	17.	1259	17.
1260	17.	1261	17.	1262	16.	1263	16.	1264	16.
1265	16.	1266	16.	1267	16.	1268	16.	1269	16.
1270	15.	1271	15.	1272	15.	1273	15.	1274	15.
1275	15.	1276	15.	1277	15.	1278	15.	1279	15.
1280	14.	1281	14.	1282	14.	1283	14.	1284	14.
1285	14.	1286	14.	1287	14.	1288	14.	1289	14.
1290	14.	1291	13.	1292	13.	1293	13.	1294	13.
1295	13.	1296	13.	1297	13.	1298	13.	1299	13.
1300	13.	1310	12.	1320	12.	1330	11.	1340	11.
1350	10.	1360	10.	1370	9.	1380	9.	1390	9.
1400	8.	1420	8.	1440	8.	1460	7.	1500	7.

Total Runoff = 35.867 Acre-Ft.
Peak Q = 188 CFS
Time to Peak Q = 1162 Minutes

APPENDIX 8
LA COUNTY STANDARD SUSMP CALCULATIONS

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-8A**

PROJECT CHARACTERISTICS

A_{Total}	6.35	
Predominate Soil Type	20	
% of Project Impervious	0.15	
% of Project Pervious	0.85	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.0	
A_p (Pervious Area)	5.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.22	0.04246	67.6	37.6	0.15	656	0.011
2	67.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.3 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3803.333** Cubic Ft.
0.09 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-9B**

PROJECT CHARACTERISTICS

A_{Total}	7.04	
Predominate Soil Type	20	
% of Project Impervious	0.66	
% of Project Pervious	0.34	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	4.6	
A_p (Pervious Area)	2.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.628	0.121204	44.8	14.8	0.66	761	0.007
2	44.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.9 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **12036.5** Cubic Ft.
0.28 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-11C**

PROJECT CHARACTERISTICS

A_{Total}	1.02	
Predominate Soil Type	20	
% of Project Impervious	0.63	
% of Project Pervious	0.37	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.6	
A_p (Pervious Area)	0.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.604	0.116572	23.1	-6.9	0.63	238	0.017
2	23.1	0.218	0.1	0.604	0.131672	21.7	-1.4	0.63	238	0.017
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.14 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1677.278** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-12C**

PROJECT CHARACTERISTICS

A_{Total}	0.63
Type of Development	Single family
Predominate Soil Type	20
% of Project Impervious	0.66
% of Project Pervious	0.34
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.4
A_p (Pervious Area)	0.2
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	23.8	-6.2	0.91	238	0.017
2	23.8	0.218	0.1	0.828	0.180504	22.4	-1.5	0.91	238	0.017
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.11 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1077.13** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-13C**

PROJECT CHARACTERISTICS

A_{Total}	0.87	
Predominate Soil Type	20	
% of Project Impervious	0.66	
% of Project Pervious	0.34	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.6	
A_p (Pervious Area)	0.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.628	0.121204	28.7	-1.3	0.66	303	0.007
2	28.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.11 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1487.465** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-14C**

PROJECT CHARACTERISTICS

A_{Total}	0.2	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	23.8	-6.2	0.91	277	0.007
2	23.8	0.218	0.1	0.828	0.180504	22.4	-1.5	0.91	277	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **450.846** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-15C**

PROJECT CHARACTERISTICS

A_{Total}	0.54	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	38.1	8.1	0.91	760	0.008
2	38.1			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1217.284** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-16C**

PROJECT CHARACTERISTICS

A_{Total}	2.78	
Predominate Soil Type	20	
% of Project Impervious	0.64	
% of Project Pervious	0.36	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.8	
A_p (Pervious Area)	1.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.612	0.118116	44.9	14.9	0.64	771	0.008
2	44.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.33 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4631.953** Cubic Ft.
0.11 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-17C**

PROJECT CHARACTERISTICS

A_{Total}	5.17	
Predominate Soil Type	20	
% of Project Impervious	0.69	
% of Project Pervious	0.31	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.6	
A_p (Pervious Area)	1.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.652	0.125836	38.5	8.5	0.69	656	0.011
2	38.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.66 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **9177.112** Cubic Ft.
0.21 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-18C**

PROJECT CHARACTERISTICS

A_{Total}	0.66	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.6	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	42.2	12.2	0.91	969	0.009
2	42.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.11 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1487.792** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-19C**

PROJECT CHARACTERISTICS

A_{Total}	0.3	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	28.5	-1.5	0.91	432	0.009
2	28.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.05 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **676.269** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-20C**

PROJECT CHARACTERISTICS

A_{Total}	4.68	
Predominate Soil Type	20	
% of Project Impervious	0.16	
% of Project Pervious	0.84	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.7	
A_p (Pervious Area)	3.9	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.228	0.044004	118.6	88.6	0.16	1749	0.005
2	118.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.21 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2905.016** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-21D**

PROJECT CHARACTERISTICS

A_{Total}	1.82	
Predominate Soil Type	20	
% of Project Impervious	0.64	
% of Project Pervious	0.36	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.2	
A_p (Pervious Area)	0.7	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.612	0.118116	36.2	6.2	0.64	495	0.008
2	36.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.22 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3032.429** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-22.1D**

PROJECT CHARACTERISTICS

A_{Total}	3.56	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.6	
A_p (Pervious Area)	0.9	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	34.3	4.3	0.74	538	0.01
2	34.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.48 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **6706.933** Cubic Ft.
0.15 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-22D**

PROJECT CHARACTERISTICS

A_{Total}	0.34
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.3
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	30.4	0.4	0.91	476	0.008
2	30.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.05 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **766.4382** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-23D**

PROJECT CHARACTERISTICS

A_{Total}	0.55	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	38.9	8.9	0.91	794	0.008
2	38.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1239.827** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-24D**

PROJECT CHARACTERISTICS

A_{Total}	0.77
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.7
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	44.8	14.8	0.91	1099	0.009
2	44.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.12 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1735.757** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-25D**

PROJECT CHARACTERISTICS

A_{Total}	0.21	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	20.7	-9.3	0.91	214	0.008
2	20.7	0.228	0.1	0.828	0.188784	18.9	-1.7	0.91	214	0.008
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **473.3883** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-26D**

PROJECT CHARACTERISTICS

A_{Total}	5.09	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.8	
A_p (Pervious Area)	1.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	46.5	16.5	0.74	1036	0.011
2	46.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.69 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **9589.407** Cubic Ft.
0.22 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-27D**

PROJECT CHARACTERISTICS

A_{Total}	0.95	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	48.8	18.8	0.91	1309	0.009
2	48.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.15 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2141.519** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-28D**

PROJECT CHARACTERISTICS

A_{Total}	1.78	
Predominate Soil Type	20	
% of Project Impervious	0.73	
% of Project Pervious	0.27	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.3	
A_p (Pervious Area)	0.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.684	0.132012	45.6	15.6	0.73	867	0.007
2	45.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.24 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3314.698** Cubic Ft.
0.08 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-29D**

PROJECT CHARACTERISTICS

A_{Total}	2.08	
Predominate Soil Type	20	
% of Project Impervious	0.14	
% of Project Pervious	0.86	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	1.8	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.212	0.040916	86.0	56.0	0.14	832	0.005
2	86.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1200.514** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-30E**

PROJECT CHARACTERISTICS

A_{Total}	0.53	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	35.8	5.8	0.91	689	0.009
2	35.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1181.756** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-31E**

PROJECT CHARACTERISTICS

A_{Total}	0.47
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.4
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	21.7	-8.3	0.91	236	0.008
2	21.7	0.228	0.1	0.828	0.188784	19.9	-1.8	0.91	236	0.008
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1059.488** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-32E**

PROJECT CHARACTERISTICS

A_{Total}	0.33
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.3
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	15.5	-14.5	0.91	109	0.006
2	15.5	0.267	0.1	0.828	0.221076	13.1	-2.4	0.91	109	0.006
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **743.8959** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-33E**

PROJECT CHARACTERISTICS

A_{Total}	5.86	
Predominate Soil Type	20	
% of Project Impervious	0.71	
% of Project Pervious	0.29	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	4.2	
A_p (Pervious Area)	1.7	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.668	0.128924	41.4	11.4	0.71	785	0.011
2	41.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.76 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **10657.17** Cubic Ft.
0.24 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-34E**

PROJECT CHARACTERISTICS

A_{Total}	5.29	
Predominate Soil Type	20	
% of Project Impervious	0.71	
% of Project Pervious	0.29	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.8	
A_p (Pervious Area)	1.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.668	0.128924	35.1	5.1	0.71	557	0.011
2	35.1			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.69 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **9620.553** Cubic Ft.
0.22 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-35E**

PROJECT CHARACTERISTICS

A_{Total}	1.23	
Predominate Soil Type	20	
% of Project Impervious	0.7	
% of Project Pervious	0.3	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.66	0.12738	39.0	9.0	0.7	626	0.008
2	39.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.16 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2210.126** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-36E**

PROJECT CHARACTERISTICS

A_{Total}	0.68	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.6	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	38.6	8.6	0.91	808	0.009
2	38.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.11 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1532.876** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-37E**

PROJECT CHARACTERISTICS

A_{Total}	1.7	
Predominate Soil Type	20	
% of Project Impervious	0.17	
% of Project Pervious	0.83	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	1.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.236	0.045548	72.3	42.3	0.17	652	0.005
2	72.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.08 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1092.267** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-38F**

PROJECT CHARACTERISTICS

A_{Total}	0.25
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.2
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	26.2	-3.8	0.91	361	0.009
2	26.2	0.206	0.1	0.828	0.170568	25.3	-0.9	0.91	361	0.009
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **563.5575** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-39F**

PROJECT CHARACTERISTICS

A_{Total}	2.03	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.5	
A_p (Pervious Area)	0.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	29.0	-1.0	0.74	400	0.012
2	29.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.27 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3824.459** Cubic Ft.
0.09 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-40F**

PROJECT CHARACTERISTICS

A_{Total}	0.05
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.0
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	12.3	-17.7	0.91	71	0.007
2	12.3	0.297	0.1	0.828	0.245916	9.9	-2.5	0.91	71	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.01 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **112.7115** Cubic Ft.
0.00 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-41F**

PROJECT CHARACTERISTICS

A_{Total}	1.79	
Predominate Soil Type	20	
% of Project Impervious	0.67	
% of Project Pervious	0.33	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.2	
A_p (Pervious Area)	0.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.636	0.122748	42.2	12.2	0.67	709	0.008
2	42.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.22 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3099.403** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-42F**

PROJECT CHARACTERISTICS

A_{Total}	0.5	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	36.3	6.3	0.91	663	0.007
2	36.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.08 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1127.115** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-43F**

PROJECT CHARACTERISTICS

A_{Total}	3.86	
Predominate Soil Type	20	
% of Project Impervious	0.68	
% of Project Pervious	0.32	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.6	
A_p (Pervious Area)	1.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.644	0.124292	52.3	22.3	0.68	1117	0.008
2	52.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.48 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **6767.699** Cubic Ft.
0.16 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-44F**

PROJECT CHARACTERISTICS

A_{Total}	2.86	
Predominate Soil Type	20	
% of Project Impervious	0.66	
% of Project Pervious	0.34	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.9	
A_p (Pervious Area)	1.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.628	0.121204	49.0	19.0	0.66	915	0.007
2	49.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.35 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4889.828** Cubic Ft.
0.11 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-45F**

PROJECT CHARACTERISTICS

A_{Total}	0.14
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.1
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	15.8	-14.2	0.91	118	0.007
2	15.8	0.259	0.1	0.828	0.214452	13.5	-2.2	0.91	118	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.03 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **315.5922** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-46F**

PROJECT CHARACTERISTICS

A_{Total}	2.68	
Predominate Soil Type	20	
% of Project Impervious	0.17	
% of Project Pervious	0.83	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	2.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.236	0.045548	93.9	63.9	0.17	1120	0.005
2	93.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.12 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1721.927** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-51G**

PROJECT CHARACTERISTICS

A_{Total}	2.99	
Predominate Soil Type	20	
% of Project Impervious	0.67	
% of Project Pervious	0.33	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.0	
A_p (Pervious Area)	1.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.636	0.122748	38.0	8.0	0.67	570	0.008
2	38.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.37 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5177.215** Cubic Ft.
0.12 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-52G**

PROJECT CHARACTERISTICS

A_{Total}	1.24	
Predominate Soil Type	20	
% of Project Impervious	0.66	
% of Project Pervious	0.34	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.8	
A_p (Pervious Area)	0.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.628	0.121204	35.1	5.1	0.66	460	0.007
2	35.1			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.15 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2120.065** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-53G**

PROJECT CHARACTERISTICS

A_{Total}	1.21	
Predominate Soil Type	20	
% of Project Impervious	0.65	
% of Project Pervious	0.35	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.8	
A_p (Pervious Area)	0.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.62	0.11966	30.6	0.6	0.65	340	0.007
2	30.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.15 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2042.42** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-54G**

PROJECT CHARACTERISTICS

A_{Total}	1.19	
Predominate Soil Type	20	
% of Project Impervious	0.65	
% of Project Pervious	0.35	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.8	
A_p (Pervious Area)	0.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.62	0.11966	30.7	0.7	0.65	344	0.007
2	30.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.14 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2008.661** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-55G**

PROJECT CHARACTERISTICS

A_{Total}	0.82	
Predominate Soil Type	20	
% of Project Impervious	0.64	
% of Project Pervious	0.36	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.612	0.118116	25.7	-4.3	0.64	235	0.007
2	25.7	0.206	0.1	0.612	0.126072	24.9	-0.9	0.64	235	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.10 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1366.259** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-56G**

PROJECT CHARACTERISTICS

A_{Total}	1.02	
Predominate Soil Type	20	
% of Project Impervious	0.69	
% of Project Pervious	0.31	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.7	
A_p (Pervious Area)	0.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.652	0.125836	34.2	4.2	0.69	485	0.009
2	34.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.13 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1810.571** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-57G**

PROJECT CHARACTERISTICS

A_{Total}	3.37
Predominate Soil Type	20
% of Project Impervious	0.68
% of Project Pervious	0.32
% of project contributing undeveloped area	0
A_I (Impervious Area)	2.3
A_p (Pervious Area)	1.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.644	0.124292	41.5	11.5	0.68	666	0.007
2	41.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.42 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5908.587** Cubic Ft.
0.14 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-58G**

PROJECT CHARACTERISTICS

A_{Total}	1.93	
Predominate Soil Type	20	
% of Project Impervious	0.73	
% of Project Pervious	0.27	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.4	
A_p (Pervious Area)	0.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.684	0.132012	51.3	21.3	0.73	1218	0.01
2	51.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.26 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3594.027** Cubic Ft.
0.08 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-59G**

PROJECT CHARACTERISTICS

A_{Total}	1.9	
Predominate Soil Type	20	
% of Project Impervious	0.52	
% of Project Pervious	0.48	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.0	
A_p (Pervious Area)	0.9	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.516	0.099588	49.3	19.3	0.52	781	0.008
2	49.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.19 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2669.139** Cubic Ft.
0.06 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-60G**

PROJECT CHARACTERISTICS

A_{Total}	9.88	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	7.3	
A_p (Pervious Area)	2.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	46.1	16.1	0.74	961	0.009
2	46.1			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.33 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **18613.62** Cubic Ft.
0.43 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-61G**

PROJECT CHARACTERISTICS

A_{Total}	1.36	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.2	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	35.4	5.4	0.91	695	0.01
2	35.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.22 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3065.753** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-62G**

PROJECT CHARACTERISTICS

A_{Total}	1.22	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.1	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	32.4	2.4	0.91	580	0.01
2	32.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.20 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2750.161** Cubic Ft.
0.06 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVE-63G**

PROJECT CHARACTERISTICS

A_{Total}	4.11	
Predominate Soil Type	20	
% of Project Impervious	0.71	
% of Project Pervious	0.29	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.9	
A_p (Pervious Area)	1.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.668	0.128924	43.7	13.7	0.71	802	0.008
2	43.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.53 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **7474.569** Cubic Ft.
0.17 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-1A**

PROJECT CHARACTERISTICS

A_{Total}	11.66	
Predominate Soil Type	20	
% of Project Impervious	0.79	
% of Project Pervious	0.21	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	9.2	
A_p (Pervious Area)	2.4	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.732	0.141276	49.5	19.5	0.79	1103	0.007
2	49.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.66 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **23236.86** Cubic Ft.
0.53 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-2A**

PROJECT CHARACTERISTICS

A_{Total}	4.08	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.0	
A_p (Pervious Area)	1.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	38.9	8.9	0.74	697	0.01
2	38.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.55 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **7686.598** Cubic Ft.
0.18 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-3A**

PROJECT CHARACTERISTICS

A_{Total}	2.39	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.2	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	51.4	21.4	0.91	1547	0.011
2	51.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.39 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5387.61** Cubic Ft.
0.12 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-4A**

PROJECT CHARACTERISTICS

A_{Total}	1.67	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.5	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	47.2	17.2	0.91	1260	0.01
2	47.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.27 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3764.564** Cubic Ft.
0.09 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-5A**

PROJECT CHARACTERISTICS

A_{Total}	0.4
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.4
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	21.0	-9.0	0.91	213	0.007
2	21.0	0.228	0.1	0.828	0.188784	19.2	-1.7	0.91	213	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.08 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **901.692** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-6A**

PROJECT CHARACTERISTICS

A_{Total}	0.78
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.7
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	24.9	-5.1	0.91	335	0.01
2	24.9	0.21	0.1	0.828	0.17388	23.8	-1.1	0.91	335	0.01
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.14 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1758.299** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-7A**

PROJECT CHARACTERISTICS

A_{Total}	0.55
Predominate Soil Type	20
% of Project Impervious	0.5
% of Project Pervious	0.5
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.3
A_p (Pervious Area)	0.3
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.5	0.0965	18.6	-11.4	0.5	100	0.008
2	18.6	0.239	0.1	0.5	0.1195	16.6	-2.0	0.5	100	0.008
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **748.6875** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-8A**

PROJECT CHARACTERISTICS

A_{Total}	1.52	
Predominate Soil Type	20	
% of Project Impervious	0.16	
% of Project Pervious	0.84	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	1.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.228	0.044004	58.1	28.1	0.16	400	0.005
2	58.1									
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **943.5096** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-9B**

PROJECT CHARACTERISTICS

A_{Total}	8.39	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	6.2	
A_p (Pervious Area)	2.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	46.6	16.6	0.74	1014	0.01
2	46.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.13 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **15806.51** Cubic Ft.
0.36 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-10B**

PROJECT CHARACTERISTICS

A_{Total}	1.03	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	36.0	6.0	0.91	650	0.007
2	36.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.17 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2321.857** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-11B**

PROJECT CHARACTERISTICS

A_{Total}	0.74	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.7	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	34.7	4.7	0.91	602	0.007
2	34.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.12 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1668.13** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-12B**

PROJECT CHARACTERISTICS

A_{Total}	9.79	
Predominate Soil Type	20	
% of Project Impervious	0.82	
% of Project Pervious	0.18	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	8.0	
A_p (Pervious Area)	1.8	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.756	0.145908	42.2	12.2	0.82	850	0.008
2	42.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.44 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **20149.88** Cubic Ft.
0.46 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-13B**

PROJECT CHARACTERISTICS

A_{Total}	1.34	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.2	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	40.7	10.7	0.91	840	0.007
2	40.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.22 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3020.668** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-14B**

PROJECT CHARACTERISTICS

A_{Total}	2.83	
Predominate Soil Type	20	
% of Project Impervious	0.71	
% of Project Pervious	0.29	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.0	
A_p (Pervious Area)	0.8	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.668	0.128924	46.5	16.5	0.71	880	0.007
2	46.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.37 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5146.723** Cubic Ft.
0.12 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVC-15C**

PROJECT CHARACTERISTICS

A_{Total}	3.54	
Predominate Soil Type	20	
% of Project Impervious	0.15	
% of Project Pervious	0.85	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	3.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.22	0.04246	65.4	35.4	0.15	560	0.008
2	65.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.15 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2120.283** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-1A**

PROJECT CHARACTERISTICS

A_{Total}	0.82	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.7	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	27.9	-2.1	0.91	383	0.007
2	27.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.13 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1848.469** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-2A**

PROJECT CHARACTERISTICS

A_{Total}	3.21	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.4	
A_p (Pervious Area)	0.8	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	27.3	-2.7	0.74	335	0.01
2	27.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.43 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **6047.544** Cubic Ft.
0.14 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-3A**

PROJECT CHARACTERISTICS

A_{Total}	1.48	
Predominate Soil Type	20	
% of Project Impervious	0.69	
% of Project Pervious	0.31	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.0	
A_p (Pervious Area)	0.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.652	0.125836	33.7	3.7	0.69	485	0.01
2	33.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.19 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2627.104** Cubic Ft.
0.06 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-4A**

PROJECT CHARACTERISTICS

A_{Total}	3.34	
Predominate Soil Type	20	
% of Project Impervious	0.64	
% of Project Pervious	0.36	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.1	
A_p (Pervious Area)	1.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.612	0.118116	38.8	8.8	0.64	550	0.007
2	38.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.40 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5565.008** Cubic Ft.
0.13 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-5B**

PROJECT CHARACTERISTICS

A_{Total}	0.34	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	23.4	-6.6	0.91	267	0.007
2	23.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.05 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **766.4382** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-6B**

PROJECT CHARACTERISTICS

A_{Total}	1.21	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	24.9	-5.1	0.74	250	0.007
2	24.9	0.21	0.1	0.692	0.14532	23.8	-1.1	0.74	250	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.18 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2279.604** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-7B**

PROJECT CHARACTERISTICS

A_{Total}	0.18	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	18.8	-11.2	0.91	221	0.018
2	18.8	0.245	0.1	0.828	0.20286	16.6	-2.2	0.91	221	0.018
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **405.7614** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-8B**

PROJECT CHARACTERISTICS

A_{Total}	0.2
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.2
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	19.0	-11.0	0.91	225	0.018
2	19.0	0.239	0.1	0.828	0.197892	17.0	-2.0	0.91	225	0.018
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **450.846** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-9B**

PROJECT CHARACTERISTICS

A_{Total}	2.22	
Predominate Soil Type	20	
% of Project Impervious	0.71	
% of Project Pervious	0.29	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.6	
A_p (Pervious Area)	0.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.668	0.128924	34.8	4.8	0.71	481	0.007
2	34.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.29 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4037.359** Cubic Ft.
0.09 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-10B**

PROJECT CHARACTERISTICS

A_{Total}	1.33	
Predominate Soil Type	20	
% of Project Impervious	0.77	
% of Project Pervious	0.23	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.0	
A_p (Pervious Area)	0.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.716	0.138188	32.9	2.9	0.77	462	0.007
2	32.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.19 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2592.582** Cubic Ft.
0.06 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-11B**

PROJECT CHARACTERISTICS

A_{Total}	0.34
Predominate Soil Type	20
% of Project Impervious	0.66
% of Project Pervious	0.34
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.2
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.628	0.121204	15.3	-14.7	0.66	85	0.008
2	15.3	0.267	0.1	0.628	0.167676	12.9	-2.4	0.66	85	0.008
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.06 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **581.3082** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-12B**

PROJECT CHARACTERISTICS

A_{Total}	3.54	
Predominate Soil Type	20	
% of Project Impervious	0.16	
% of Project Pervious	0.84	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.6	
A_p (Pervious Area)	3.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.228	0.044004	102.4	72.4	0.16	1360	0.006
2	102.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.16 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2197.384** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-12C**

PROJECT CHARACTERISTICS

A_{Total}	0.87
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.8
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	44.3	14.3	0.91	1000	0.007
2	44.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.14 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1961.18** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-13C**

PROJECT CHARACTERISTICS

A_{Total}	3.6	
Predominate Soil Type	20	
% of Project Impervious	0.65	
% of Project Pervious	0.35	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.3	
A_p (Pervious Area)	1.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.62	0.11966	40.1	10.1	0.65	597	0.007
2	40.1			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.43 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **6076.62** Cubic Ft.
0.14 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-14C**

PROJECT CHARACTERISTICS

A_{Total}	2.58	
Predominate Soil Type	20	
% of Project Impervious	0.68	
% of Project Pervious	0.32	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.8	
A_p (Pervious Area)	0.8	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.644	0.124292	52.9	22.9	0.68	1147	0.008
2	52.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.32 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4523.488** Cubic Ft.
0.10 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-15C**

PROJECT CHARACTERISTICS

A_{Total}	2.13	
Predominate Soil Type	20	
% of Project Impervious	0.65	
% of Project Pervious	0.35	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.4	
A_p (Pervious Area)	0.7	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.62	0.11966	40.4	10.4	0.65	650	0.009
2	40.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.26 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3595.334** Cubic Ft.
0.08 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-16C**

PROJECT CHARACTERISTICS

A_{Total}	0.99	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	30.6	0.6	0.91	466	0.007
2	30.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.16 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2231.688** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-17C**

PROJECT CHARACTERISTICS

A_{Total}	1.91	
Predominate Soil Type	20	
% of Project Impervious	0.16	
% of Project Pervious	0.84	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	1.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.228	0.044004	81.3	51.3	0.16	800	0.005
2	81.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.08 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1185.594** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-18D**

PROJECT CHARACTERISTICS

A_{Total}	1.93	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.4	
A_p (Pervious Area)	0.5	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	33.3	3.3	0.74	517	0.011
2	33.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.26 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **3636.062** Cubic Ft.
0.08 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-19D**

PROJECT CHARACTERISTICS

A_{Total}	0.51	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.5	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	25.0	-5.0	0.91	305	0.007
2	25.0	0.21	0.1	0.828	0.17388	23.9	-1.1	0.91	305	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1149.657** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-20D**

PROJECT CHARACTERISTICS

A_{Total}	0.38
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.3
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	25.2	-4.8	0.91	310	0.007
2	25.2	0.21	0.1	0.828	0.17388	24.1	-1.1	0.91	310	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **856.6074** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-21D**

PROJECT CHARACTERISTICS

A_{Total}	11
Predominate Soil Type	20
% of Project Impervious	0.74
% of Project Pervious	0.26
% of project contributing undeveloped area	0
A_I (Impervious Area)	8.1
A_p (Pervious Area)	2.9
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	58.4	28.4	0.74	1570	0.009
2	58.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.48 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **20723.67** Cubic Ft.
0.48 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-22D**

PROJECT CHARACTERISTICS

A_{Total}	1.88	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.7	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	47.5	17.5	0.91	1200	0.008
2	47.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.30 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4237.952** Cubic Ft.
0.10 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-23D**

PROJECT CHARACTERISTICS

A_{Total}	1.26	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.1	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	44.2	14.2	0.91	1071	0.009
2	44.2			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.20 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2840.33** Cubic Ft.
0.07 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-24D**

PROJECT CHARACTERISTICS

A_{Total}	4.91	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.6	
A_p (Pervious Area)	1.3	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	46.3	16.3	0.74	1000	0.01
2	46.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D * I_X * A_{Total} * (1.008333 ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.66 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac}) * [(A_I)(0.9) + (A_p + A_u)(C_u)]$

V_{m+} **9250.293** Cubic Ft.
0.21 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-25D**

PROJECT CHARACTERISTICS

A_{Total}	0.49	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.4	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	25.5	-4.5	0.91	320	0.007
2	25.5	0.21	0.1	0.828	0.17388	24.4	-1.1	0.91	320	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1104.573** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-26.1D**

PROJECT CHARACTERISTICS

A_{Total}	0.56
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.5
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	23.9	-6.1	0.91	280	0.007
2	23.9	0.214	0.1	0.828	0.177192	22.7	-1.2	0.91	280	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.10 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1262.369** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-26D**

PROJECT CHARACTERISTICS

A_{Total}	1.11	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.0	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	20.3	-9.7	0.91	220	0.01
2	20.3	0.233	0.1	0.828	0.192924	18.4	-1.9	0.91	220	0.01
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.22 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2502.195** Cubic Ft.
0.06 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-27D**

PROJECT CHARACTERISTICS

A_{Total}	1.02	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.9	
A_p (Pervious Area)	0.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	26.3	-3.7	0.91	340	0.007
2	26.3	0.206	0.1	0.828	0.170568	25.4	-0.9	0.91	340	0.007
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.18 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **2299.315** Cubic Ft.
0.05 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-28D**

PROJECT CHARACTERISTICS

A_{Total}	17.71	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	13.1	
A_p (Pervious Area)	4.6	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	45.9	15.9	0.74	982	0.01
2	45.9			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **2.38 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **33365.11** Cubic Ft.
0.77 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-29D**

PROJECT CHARACTERISTICS

A_{Total}	4.06	
Predominate Soil Type	20	
% of Project Impervious	0.74	
% of Project Pervious	0.26	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	3.0	
A_p (Pervious Area)	1.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.692	0.133556	37.3	7.3	0.74	637	0.01
2	37.3			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.55 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **7648.918** Cubic Ft.
0.18 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-31E**

PROJECT CHARACTERISTICS

A_{Total}	2.17	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.0	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	37.8	7.8	0.91	720	0.007
2	37.8			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.35 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4891.679** Cubic Ft.
0.11 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-32E**

PROJECT CHARACTERISTICS

A_{Total}	0.7
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.6
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	24.4	-5.6	0.91	385	0.019
2	24.4	0.21	0.1	0.828	0.17388	23.4	-1.0	0.91	385	0.019
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.12 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1577.961** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-33E**

PROJECT CHARACTERISTICS

A_{Total}	1.99	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	1.8	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	30.6	0.6	0.91	615	0.019
2	30.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.32 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **4485.918** Cubic Ft.
0.10 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-34E**

PROJECT CHARACTERISTICS

A_{Total}	2.44	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	2.2	
A_p (Pervious Area)	0.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	39.7	9.7	0.91	880	0.01
2	39.7			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.39 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **5500.321** Cubic Ft.
0.13 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-35E**

PROJECT CHARACTERISTICS

A_{Total}	0.65
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.6
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	31.4	1.4	0.91	510	0.008
2	31.4									
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.10 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1465.25** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-36E**

PROJECT CHARACTERISTICS

A_{Total}	0.26
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.2
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	18.4	-11.6	0.91	180	0.01
2	18.4	0.245	0.1	0.828	0.20286	16.3	-2.1	0.91	180	0.01
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.05 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **586.0998** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-37E**

PROJECT CHARACTERISTICS

A_{Total}	0.37
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.3
A_p (Pervious Area)	0.0
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	24.0	-6.0	0.91	320	0.011
2	24.0	0.214	0.1	0.828	0.177192	22.8	-1.3	0.91	320	0.011
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **834.0651** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-38E**

PROJECT CHARACTERISTICS

A_{Total}	0.77
Predominate Soil Type	20
% of Project Impervious	0.91
% of Project Pervious	0.09
% of project contributing undeveloped area	0
A_I (Impervious Area)	0.7
A_p (Pervious Area)	0.1
A_u (Contrib Undeveloped upstream area)	0

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	28.0	-2.0	0.91	460	0.013
2	28.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.12 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1735.757** Cubic Ft.
0.04 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-39.1E**

PROJECT CHARACTERISTICS

A_{Total}	0.41	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.4	
A_p (Pervious Area)	0.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	31.6	1.6	0.91	550	0.01
2	31.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **924.2343** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-39E**

PROJECT CHARACTERISTICS

A_{Total}	5.72	
Predominate Soil Type	20	
% of Project Impervious	0.79	
% of Project Pervious	0.21	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	4.5	
A_p (Pervious Area)	1.2	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.732	0.141276	43.4	13.4	0.79	930	0.01
2	43.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.81 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **11399.22** Cubic Ft.
0.26 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-40E**

PROJECT CHARACTERISTICS

A_{Total}	7.41	
Predominate Soil Type	20	
% of Project Impervious	0.91	
% of Project Pervious	0.09	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	6.7	
A_p (Pervious Area)	0.7	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.828	0.159804	39.6	9.6	0.91	1050	0.019
2	39.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **1.19 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **16703.84** Cubic Ft.
0.38 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-41E**

PROJECT CHARACTERISTICS

A_{Total}	2.32	
Predominate Soil Type	20	
% of Project Impervious	0.13	
% of Project Pervious	0.87	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.3	
A_p (Pervious Area)	2.0	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.204	0.039372	82.4	52.4	0.13	730	0.005
2	82.4			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.09 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **1288.505** Cubic Ft.
0.03 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-42E**

PROJECT CHARACTERISTICS

A_{Total}	1.04	
Predominate Soil Type	20	
% of Project Impervious	0.13	
% of Project Pervious	0.87	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.1	
A_p (Pervious Area)	0.9	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.204	0.039372	61.6	31.6	0.13	400	0.005
2	61.6			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.04 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **577.6056** Cubic Ft.
0.01 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-43G**

PROJECT CHARACTERISTICS

A_{Total}	1.29	
Predominate Soil Type	20	
% of Project Impervious	0.15	
% of Project Pervious	0.85	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	1.1	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.22	0.04246	68.5	38.5	0.15	540	0.005
2	68.5			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.06 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **772.6455** Cubic Ft.
0.02 Acre ft

SUSMP CALCULATIONS

PROJECT: **LANDMARK VILLAGE**
SUB-BASIN NUMBER **RVW-44G**

PROJECT CHARACTERISTICS

A_{Total}	1.94	
Predominate Soil Type	20	
% of Project Impervious	0.1	
% of Project Pervious	0.9	
% of project contributing undeveloped area	0	
A_I (Impervious Area)	0.2	
A_p (Pervious Area)	1.7	
A_u (Contrib Undeveloped upstream area)	0	

TABLE FOR T_c ITERATIONS

Iteration No.	Initial Tc (min)	I_X (in/hr)	C_U	C_D	C_D*I_X (in/hr	Calculated Tc (min)	Difference (min)	Imp.	Length	Slope
1	30	0.193	0.1	0.18	0.03474	29.0	-1.0	0.1	140	0.05
2	29.0			0	0	#DIV/0!	#DIV/0!			#DIV/0!
3										

Acceptable Tc Value 30.0 Minutes. Use 30 min max for Tc.

Peak Mitigation Flow Rate Q_{pm}

$Q_{pm} = C_D*I_X*A_{Total}*(1.008333ft^3\text{-hour/acre-inches-second})$

Q_{PM} **0.07 cfs.**

Mitigation Volume Calculations - V_m

$V_m = 2722.5(\text{Cu Ft/Ac})*[(A_I)(0.9) + (A_p+A_u)(C_u)]$

V_{m+} **950.697** Cubic Ft.
0.02 Acre ft

APPENDIX 9
GEOSYNTEC MEMORANDUM

Stormwater Treatment Control
BMP Sizing

Geosyntec consultants

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Los Angeles, California 90064
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FAX 310.839.6041
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Memorandum

Date:	22 October 2009
To:	Nicholas Khoo, Erik Winata, John Vlassis, and Ross Barker, Psomas
Copy to:	Fred MacMurdo, Newhall Land
From:	Lisa Austin, Nichole Dunn and Aaron Poresky, Geosyntec Consultants
Subject:	Landmark Village - Evaluation of Stormwater Treatment Control BMP Sizing

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The purpose of this memorandum is to verify that the treatment control BMPs shown on the *Newhall Ranch - Landmark Village Drainage Concept Vesting Tentative Tract Map No. 53108 SUSMP Plan* (Rev: April 2008) prepared by Psomas are consistent with the *Landmark Village Water Quality Technical Report* (Geosyntec, March 2008).

TREATMENT CONTROL BMPS

The stormwater treatment control BMPs for the Landmark Village Project consist of one extended detention (ED) basin, eleven bioretention areas, one vegetated swale, and two modular wetlands. The ED basin, identified as BMP-8, will provide treatment for 57 acres (21%) of the project development area; the 11 bioretention areas will provide treatment for 204 acres (75%) of the project development area; the vegetated swale will provide treatment for 3.54 acres (~1.5%) of the project development area; and two modular wetlands will provide treatment for 2.6 acres (~1%) of the project development area.

The applicable sizing criterion for the project's ED basin and bioretention areas is the capture and treatment of 80 percent of the average annual runoff volume, with a drawdown time of 48 hours for the ED basin. This sizing criterion utilizes historical rainfall data with continuous simulation modeling and is consistent with volume-based sizing criterion #2 from the SUSMP Manual. The applicable sizing criterion for the Project's vegetated swale and modular wetlands uses a rainfall intensity based on SUSMP Appendix A, with a minimum of 0.3 inches per hour, which meets and exceeds flow-based sizing criterion #1 from the SUSMP Manual.

VERIFICATION OF BMP SIZING

To verify that the treatment BMPs provided on the *Newhall Ranch - Landmark Village Drainage Concept Vesting Tentative Tract Map No. 53108 SUSMP Plan* (Rev: April 2008) are adequately sized, catchment delineations, catchment characteristics, and water quality volume and flow calculations were reviewed.

The ED basin for the Project is listed in Table 1 below along with its tributary area, Newhall Land volume, and the volume of the basin provided on the Drainage Concept SUSMP Plan. As shown in Table 1, the basin volume provided is greater than the minimum mitigation volume for 80% capture and is consistent with the sizing in the *Landmark Village Water Quality Technical Report*.

Table 1: Extended Detention Basin Sizing Results

BMP ID	Tributary Area ID(s)	Tributary Area (acres)	Catchment Imperviousness (%) [1]	Newhall Land Basin Volume Required[2] (ac-ft)	Basin Volume Provided[3] (ac-ft)
BMP-8	RVC-1A, RVC-2A, RVC-3A, RVC-4A, RVC-5A, RVC-6A, RVC-7A, RVC-9B, RVC-10B, RVC-11B, RVC-12B, RVC-13B, RVC-14B, RVW-1A, RVW-2A, RVW-3A, RVW-4A, RVW-5B, RVW-6B, RVW-7B, RVW-8B, RVW-9B, RVW-10B, RVW-11B, RVW-12B, RVW-17C, RVW-12C, RVW-13C, RVW-14C, RVW-15C, RVW-16C, RVW-43G	57	76	4.6	5.06

[1] Im -based values per the *Landmark Village Tentative Tract Map 53108 Drainage Concept* (Psomas, April 2008).
[2] Basin sized using SWMM catchment-specific modeling results to capture and treat 80% of annual average runoff. Design volume does not account for additional storage needed for sediment storage and freeboard.
[3] Basin sizing per the *Newhall Ranch - Landmark Village Vesting Tentative Tract Map 53108 Drainage Concept SUSMP Plan* (Psomas, April 2008).

The bioretention area BMPs for the Project are listed in Table 2 below along with their tributary area, Newhall Land required volume and area, and the area provided on the Drainage Concept

SUSMP Plan. As shown in Table 2, the bioretention areas provided are equal to or greater than the minimum mitigation area for 80% capture and are consistent with the sizing in the *Landmark Village Water Quality Technical Report*.

Table 2: Bioretention Sizing Results

BMP ID	Tributary Area ID(s)	Tributary Area (acres)	Catchment Imperviousness (%) [1]	Newhall Land Volume Required [2] (acre-ft)	Newhall Land Area Required [3] (acre)	Area Provided [4] (acre)
BMP-1	RVE-20C, RVE-9B	11.7	46	0.39	0.26	1.18
BMP-2	RVE-11C, RVE-12C, RVE-13C, RVE-14C, RVE-15C, RVE-16C, RVE-17C, RVE-18C, RVE-19C, RVE-29D	14.3	62	0.58	0.38	0.63
BMP-3	RVE-21D, RVE-22.1D, RVE-22D, RVE-23D, RVE-24D, RVE-25D, RVE-26D, RVE-27D, RVE-28D, RVE-37E	16.8	70	0.74	0.49	0.54
BMP-4	RVE-30E, RVE-31E, RVE-32E, RVE-33E, RVE-34E, RVE-35E, RVE-36E, RVE-46F	17.1	65	0.71	0.48	1.08
BMP-5	RVC-8A, RVE-38F, RVE-39F, RVE-40F, RVE-41F, RVE-42F, RVE-43F, RVE-44F, RVE-45F	13.0	64	0.54	0.36	0.41

BMP ID	Tributary Area ID(s)	Tributary Area (acres)	Catchment Imperviousness (%) [1]	Newhall Land Volume Required [2] (acre-ft)	Newhall Land Area Required [3] (acre)	Area Provided [4] (acre)
BMP-6/BMP-7	RVC-1A, RVC-2A, RVC-3A, RVC-4A, RVC-5A, RVC-6A, RVC-7A, RVC-9B, RVC-10B, RVC-11B, RVC-12B, RVC-13B, RVC-14B, RVW-1A, RVW-2A, RVW-3A, RVW-4A, RVW-5B, RVW-6B, RVW-7B, RVW-8B, RVW-9B, RVW-10B, RVW-11B, RVW-12B, RVW-17C, RVW-12C, RVW-13C, RVW-14C, RVW-15C, RVW-16C, RVW-43G	77.2	72	3.45	2.30	2.73
BMP-9	RVW-32E, RVW-37E, RVW-38E, RVW-40E, RVW-42F	9.6	83	0.47	0.32	0.34
BMP-10	RVW-39.1E, RVW-39E, RVW-41E	8.5	61	0.34	0.23	0.68
BMP-11/BMP-12	RVE-51G, RVE-52G, RVE-53G, RVE-54G, RVE-55G, RVE-56G, RVE-57G, RVE-58G, RVE-59G, RVE-63G	19.8	67	0.84	0.56	0.81
BMP-14	RVE-60G	9.9	74	0.45	0.30	0.30
BMP-17	RVE-8A	6.4	15	0.13	0.09	0.20

[1] Im -based values per the *Landmark Village Tentative Tract Map 53108 Drainage Concept* (Psomas, April 2008).
[2] Bioretention volume based on maximum bioretention depth of 18 inches.
[3] Bioretention area sized to capture and treat 80% of annual average runoff.
[4] Bioretention area per the *Newhall Ranch - Landmark Village Vesting Tentative Tract Map 53108 Drainage Concept SUSMP Plan* (Psomas, April 2008).

The Newhall Land Area Required is the wetted BMP area illustrated in Figure 1 below. The Newhall Land Area Required is equivalent to the total footprint required when the bioretention

side slopes are vertical. Figure 2 below illustrates a bioretention cross section that has sloped side walls; this figure illustrates the increase the total area required with side slopes.

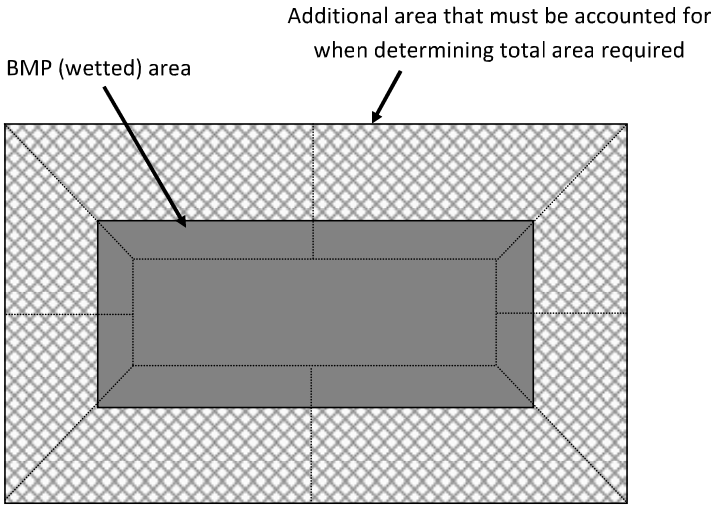


Figure 1: Plan view of a rectangular shaped Bioretention facility, illustrating BMP "wetted" area

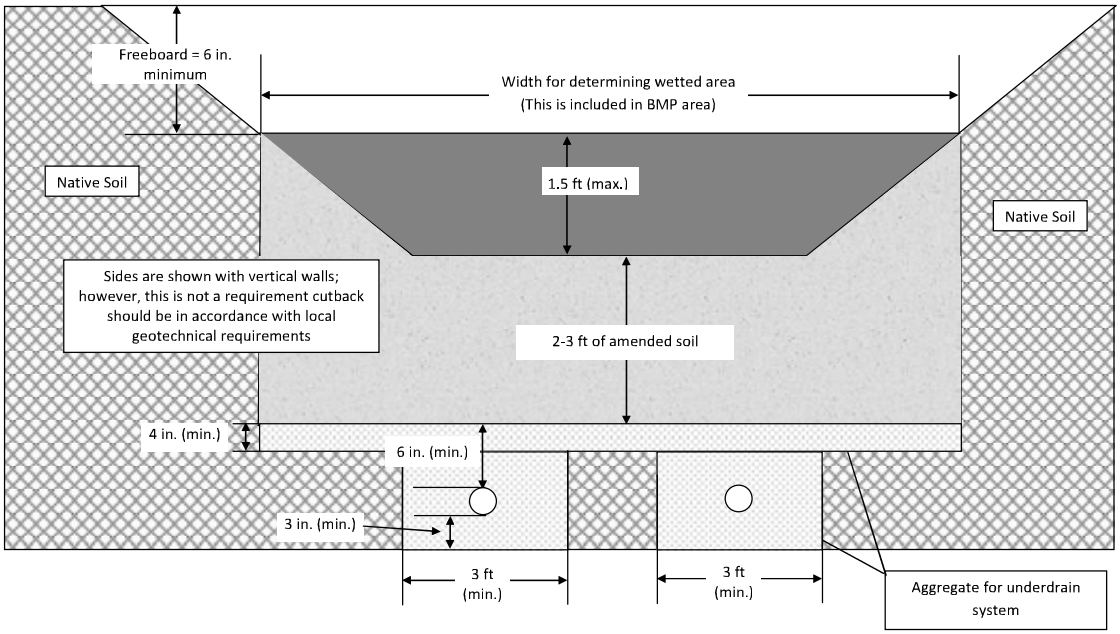


Figure 2: Bioretention area cross-section schematic, shown with side slopes above ground level and vertical walls below ground level

The vegetated swale for the Project is listed in Table 3 below, along with the tributary area, the design flow rate, and the design dimensions provided, including longitudinal slope, length, bottom width and side slopes. The rainfall intensity used in the sizing calculation was 0.359 in/hr, which meets or exceeds the 0.3 in/hr Newhall Land minimum flow-based sizing criteria. Swale design criterion is based on a 10-minute retention time. As shown in Table 3, the vegetated swale design achieves the 10-minute retention time criterion when sized with rainfall intensities, which meet or exceed the Newhall Land flow-based sizing criteria, and this design is consistent with the sizing in the *Landmark Village Water Quality Technical Report*.

Table 3: Vegetated Swale Sizing Results

BMP ID	Tributary Area RVE-15C (acres)	Catchment Imper-viousness [1] (%)	Newhall Land Design Flow Rate[2] (cfs)	Swale Dimensions Provided [3]				Achieves 10-min retention time [4]
				Longi-tudinal Slope (%)	Length (ft)	Bottom Width (ft)	Side Slopes (H:V)	
BMP-13	3.54	15	0.31	1	330	3	4:1	yes

[1] Im -based values per the *Landmark Village Tentative Tract Map 53108 Drainage Concept* (Psomas, April 2008).

[2] Design flow rate using a rainfall intensity based on Los Angeles County SUSMP Manual Appendix A, with a minimum of 0.3 in/h .

[3] Swale dimensions per the *Newhall Ranch - Landmark Village Vesting Tentative Tract Map 53108 Drainage Concept SUSMP Plan* (Psomas, April 2008).

[4] Assumes a head feed swale configuration and a Manning's n of 0.25.

The modular wetlands for the Project are listed in Table 4 below, along with the tributary area, the design flow rate, and the number of units required and provided at each location. Rainfall intensities used in the sizing calculations were 0.359 in/hr for BMP-15 and 0.339 in/hr for BMP-16, depending on catchment area, which in all cases meets or exceeds the 0.3 in/hr Newhall Land criteria. Modular wetlands design criteria are based on a MWS-Linear Curb and Grate Type Modular Wetlands™ system with outer dimensions of 22' x 5' x 4.8' providing a peak treatment capacity of 0.27 cfs.

Table 4: Modular Wetland Sizing Results

BMP ID	Tributary Area ID(s)	Tributary Area (acres)	Catchment Imperviousness (%) [1]	Newhall Land Design Flow Rate[2] (cfs)	Newhall Land Design Flow Rate (gpm)	Number of Units Required
BMP-15	RVE-62G	1.22	91	0.37	165	2
BMP-16	RVE-61G	1.36	91	0.38	173	2

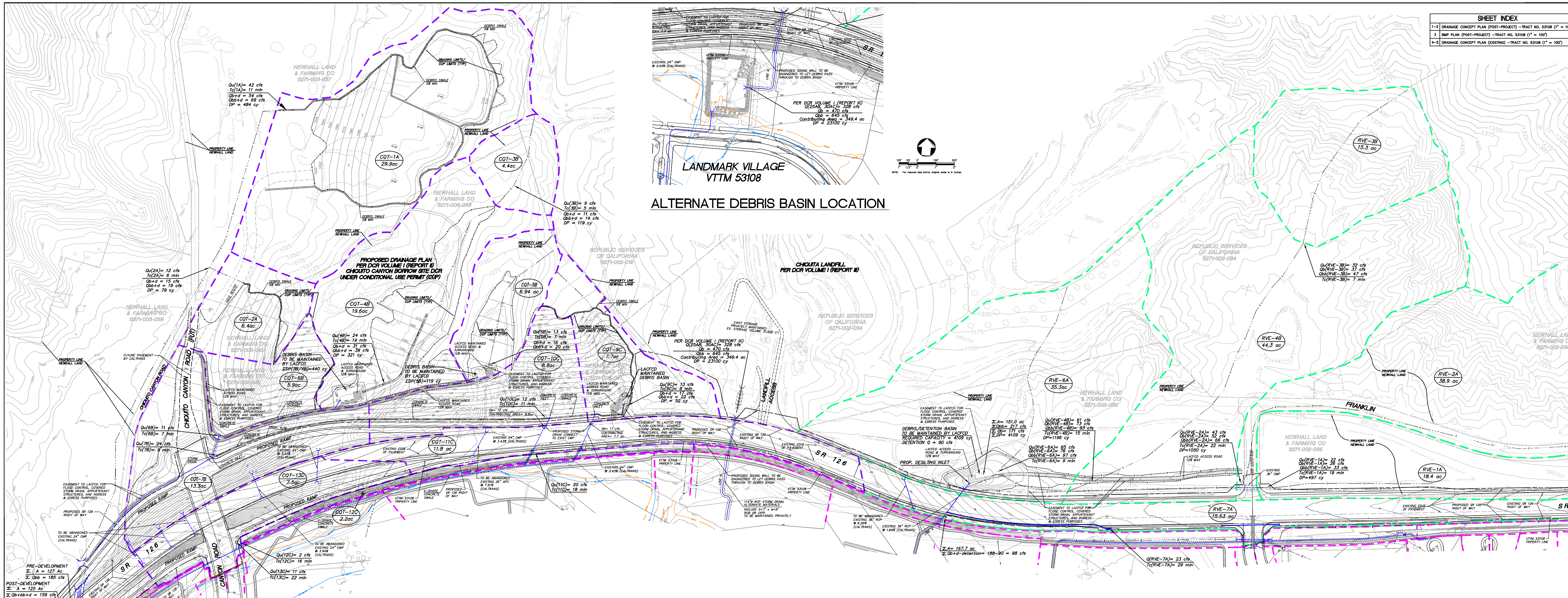
[1] Im -based values per the *Landmark Village Tentative Tract Map 53108 Drainage Concept* (Psomas, April 2008).

[2] Design flow rate using a rainfall intensity based on Los Angeles County SUSMP Manual Appendix A, with a minimum of 0.3 in/hr per Newhall Land criteria.

[3] Number of units required based on using a MWS-Linear Curb and Grate Type modular wetland system with a peak treatment capacity of 0.27 cfs (121 gpm) with outer dimensions of 22' x 5' x 4.8' to treat area specified per the *Newhall Ranch - Landmark Village Vesting Tentative Tract Map 53108 Drainage Concept SUSMP Plan* (Psomas, April 2008).

As shown in Table 4, the modular wetland units will meet or exceed the Newhall Land sizing criteria and are consistent with the sizing in the *Landmark Village Water Quality Technical Report*.

APPENDIX 10
DRAINAGE CONCEPT EXHIBITS



SHEET INDEX	
1-2	DRAINAGE CONCEPT PLAN (POST-PROJECT) -TRACT NO. 53108 (1" = 100')
3	BMP PLAN (POST-PROJECT) -TRACT NO. 53108 (1" = 100')
4-5	DRAINAGE CONCEPT PLAN (EXISTING) -TRACT NO. 53108 (1" = 100')

DRAINAGE CONCEPT NOTES:

1. HYDROLOGY INFORMATION AND STORM DRAIN ALIGNMENTS SHOWN ARE NOT NECESSARILY APPROVED.
2. ALL STREET DRAINAGE REQUIREMENTS WILL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
5. APPROVAL OF THE DRAINAGE CONCEPT DOES NOT CONSTITUTE DETERMINATION THAT THE OFFSITE IMPROVEMENTS ARE REQUIRED WITHIN THE MEANING OF GOVERNMENT CODE SECTION 66462.5, (EXCEPT AS NOTED).
6. DELINEATE THE FLOOD HAZARD AREA AND DEDICATE TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT BUILDING WITHIN THAT AREA ON THE FINAL MAP.
7. THE PROPOSED WATER QUALITY BASIN, SWALES, DEBRIS BASINS AND STORM DRAIN FACILITIES WILL BE MAINTAINED PUBLICLY. UNLESS OTHERWISE SHOWN. ALSO, PROPOSED DRAINAGE FACILITIES WITHIN CALTRANS R/W SHALL BE MAINTAINED BY CALTRANS
8. BECAUSE THE WATER QUALITY BASIN, AND SWALES PROVIDE AN OUTLET, AND NO PERCOLATION CONSIDERED, A 7-DAY PERCOLATION TEST WILL NOT BE REQUIRED.
9. FLOODWAY MAP WILL BE AMENDED AS PART OF THE RIVER ANALYSIS BY PACE ENGINEERING; PER DCR VOLUME V OF V.
10. CLOMR UNDER PREPARATION BY PACE; PER DCR VOLUME V OF V.
11. WHEN DUPLICATE DATA SHOWN ON BOTH TABLE & PLAN, PLAN DATA SHOWN FOR INFORMATIONAL PURPOSES ONLY.
12. ALL RIVER SWALES TO HAVE EMERGENCY OVERFLOW SPILLWAY FOR FLOOD DISCHARGE.
13. ENCROACHMENT INTO EXISTING MAIN LINE FLOODWAY AND FEMA ZONE "A" WILL BE MITIGATED.
14. AN ASSESSMENT DISTRICT SHALL BE FORMED TO FINANCE THE FUTURE ONGOING MAINTENANCE AND CAPITAL REPLACEMENT OF ALL DRAINAGE DEVICES/SYSTEMS IDENTIFIED BY THE PUBLIC WORKS. THE SUBDIVIDER SHALL DEPOSIT THE FIRST YEAR'S TOTAL ASSESSMENT BASED ON THE PUBLIC WORKS ENGINEERING REPORT. THIS WILL FUND THE FIRST YEAR'S MAINTENANCE AFTER THE FACILITIES ARE ACCEPTED. THE SECOND AND SUBSEQUENT YEARS' ASSESSMENT WILL BE COLLECTED THROUGH THE PROPERTY TAX BILL. THIS IS REQUIRED TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.

PROJECT SITE

VICINITY MAP
NOT TO SCALE

LEGEND:

110A — SUB-BASIN NUMBER
007.3 ac — SUB-BASIN AREA (ac)

PROJECT BOUNDARY
LANDMARK ONSITE DRAINAGE BOUNDARY

Q	Q unburned/develop
Qd	Q developed
Qb	Q burned
Qbb	Q burned+bulked
DP	DEBRIS PRODUCTION

DRAINAGE DESIGN CRITERIA:

1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 58% MULTI-FAMILY - 74% COMMERCIAL - 91% SCHOOL - 82% OPEN SPACE - 0% PARK - 15% ROADWAY - 91%
5. DPA= 09
6. BULK FACTOR= 1.27

IS CONCEPTUALLY APPROVED

OFFSITE TRIBUTARY AREA

REV: NOVEMBER 12, 2009
DATE: SEPT. 15, 2004

LEGAL DESCRIPTION:
PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

PSOMAS
28470 Avenue Stanford, Suite 300
Santa Clarita, CA 91355-1460
(661) 219-6000 (661) 775-2718 (FAX)
ERIK WINATA R.C.E. NO. C67285 DATE

DESIGNED: NSK/EW/JJ
DRAFTED: TC/MH
CHECKED: EW/RB/JV

OWNER/DEVELOPER:
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE: MR. FRED MACMURDO

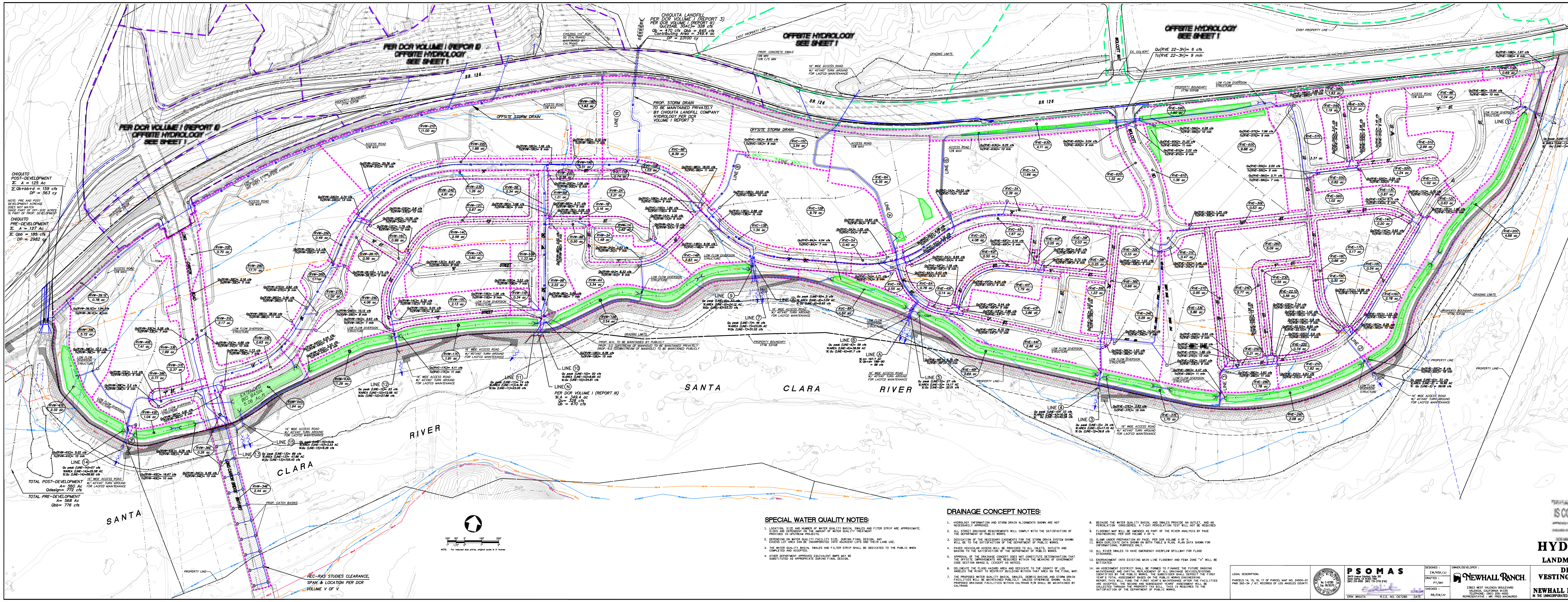
DRAINAGE CONCEPT PLAN FOR VESTING TENTATIVE TRACT MAP NO. 53108
NEWHALL RANCH - LANDMARK VILLAGE

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 1 OF 5

RV062-01

Plotted - 11/12/2009 9:57:02 AM :: Saved - 11/11/2009 3:02:06 PM :: V:\Newhall\PRODDATA\53108-MP\ENGR\MASTDES\VTTM\PI\RV062-01.dwg :: ewinata



DRAINAGE DESIGN CRITERIA:

1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020 & 097
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 58%, MULTI-FAMILY - 74%, COMMERCIAL - 91%, SCHOOL - 82%, OPEN SPACE - 0%, PARK - 15%, ROADWAY - 91%
5. DPA= 09
6. BULK FACTOR= 1.27

SPECIAL WATER QUALITY NOTES:

1. LOCATION, SIZE AND NUMBER OF WATER QUALITY BASIN, SWALES AND FITER STRIP ARE APPROXIMATE. SIZES ARE DEPENDENT ON THE AMOUNT OF WATER QUALITY TREATMENT PROVIDED IN UPSTREAM PROJECTS.
2. DEPENDING ON WATER QUALITY FACILITY SIZE, DURING FINAL DESIGN, ANY EXCESS LOT AREA CAN BE INCORPORATED INTO ADJACENT LOTS AND THEIR LAND USE.
3. THE WATER QUALITY BASIN, SWALES AND FILTER STRIP SHALL BE DEDICATED TO THE PUBLIC WHEN COMPLETED AND ACCEPTED.
4. OTHER DEPARTMENT APPROVED EQUIVALENT BMPS MAY BE SUBSTITUTED AS APPROPRIATE DURING FINAL DESIGN.

DRAINAGE CONCEPT NOTES:

1. HYDROLOGY INFORMATION AND STORM DRAIN ALIGNMENTS SHOWN ARE NOT NECESSARILY APPROVED.
2. ALL STREET DRAINAGE REQUIREMENTS WILL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
5. APPROVAL OF THE DRAINAGE CONCEPT DOES NOT CONSTITUTE DETERMINATION THAT THE OFFSITE IMPROVEMENTS ARE REQUIRED WITHIN THE MEANING OF GOVERNMENT CODE SECTION 66462.5, (EXCEPT AS NOTED).
6. DELINEATE THE FLOOD HAZARD AREA AND DEDICATE TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT BUILDING WITHIN THAT AREA ON THE FINAL MAP.
7. THE PROPOSED WATER QUALITY BASIN, SWALES, DEBRIS BASINS AND STORM DRAIN FACILITIES WILL BE MAINTAINED PUBLICLY. UNLESS OTHERWISE SHOWN. ALSO, PROPOSED DRAINAGE FACILITIES WITHIN CALTRANS R/W SHALL BE MAINTAINED BY CALTRANS
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IS CONCEPTUALLY APPROVED

HYDROLOGY PLAN

LANDMARK PROJECT AREA

REV: NOVEMBER 12, 2009
DATE: SEPT. 15, 2004

LEGAL DESCRIPTION:

PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28470 Avenue Stanford, Suite 300
Santa Clarita, CA 91355-1460
(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C67285 DATE

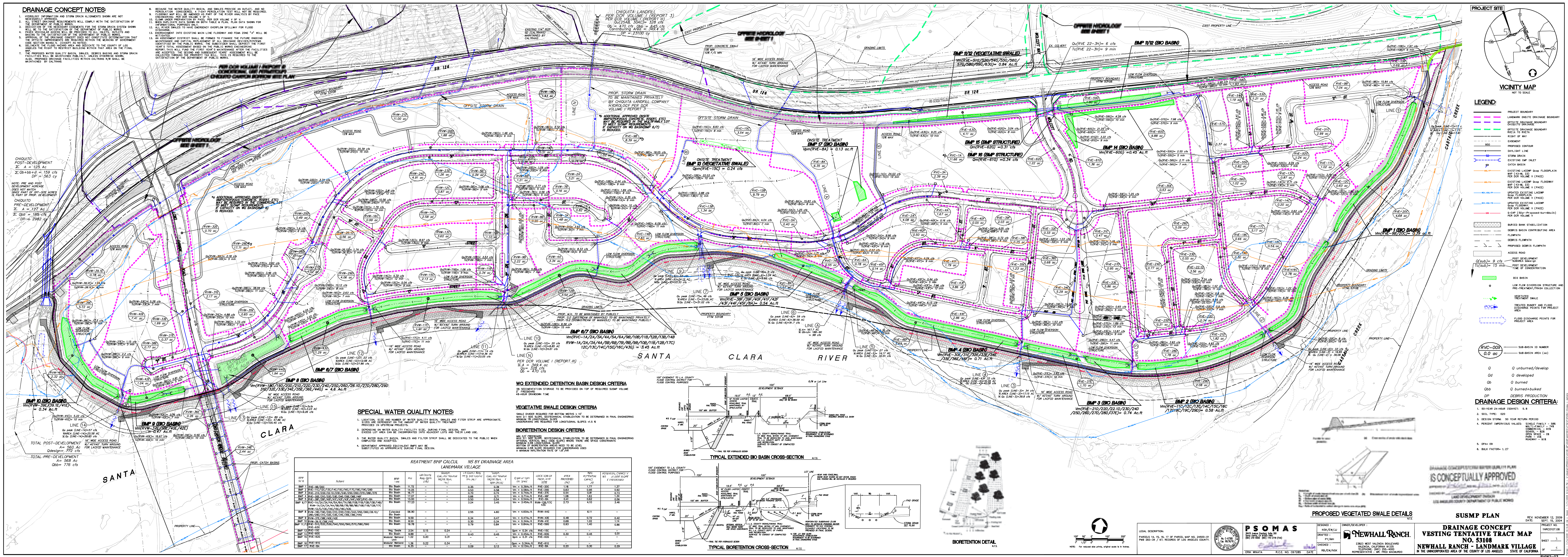
DESIGNED: EW/NSK/JJ
DRAFTED: PY/MH
CHECKED: RB/EW/JV

OWNER/DEVELOPER:
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE: MR. FRED MACMURDO

DRAINAGE CONCEPT
VESTING TENTATIVE TRACT MAP
NO. 53108
NEWHALL RANCH - LANDMARK VILLAGE
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 2 OF 5

RV062-02


DRAINAGE CONCEPT NOTES:
SPECIAL WATER QUALITY NOTES:
WQ EXTENDED DETENTION BASIN DESIGN CRITERIA
VEGETATIVE SWALE DESIGN CRITERIA
BIORETENTION DESIGN CRITERIA
TYPICAL EXTENDED BIO BASIN CROSS-SECTION
TYPICAL BIORETENTION CROSS-SECTION
BIORETENTION DETAIL
PROPOSED VEGETATED SWALE DETAILS
N.T.S.
SANTA CLARA RIVER
CASTAIC CREEK
SR 126
BMP 1 (BIO BASIN)
BMP 2 (BIO BASIN)
BMP 3 (BIO BASIN)
BMP 4 (BIO BASIN)
BMP 5 (BIO BASIN)
BMP 6/7 (BIO BASIN)
BMP 8 (BIO BASIN)
BMP 9 (BIO BASIN)
BMP 10 (BIO BASIN)
BMP 11/12 (VEGETATIVE SWALE)
BMP 13 (VEGETATIVE SWALE)
BMP 14 (BIO BASIN)
BMP 15 (BMP STRUCTURE)
BMP 16 (BMP STRUCTURE)
BMP 17 (BIO BASIN)
OFFSITE HYDROLOGY SEE SHEET 1
PROJECT SITE
VICINITY MAP
NOT TO SCALE
LEGEND:
DRAINAGE DESIGN CRITERIA:
1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 58% MULTI-FAMILY - 74% COMMERCIAL - 91% SCHOOL - 82% OPEN SPACE - 0% PARK - 15% ROADWAY - 91%
5. DPA= 09
6. BULK FACTOR= 1.27
IS CONCEPTUALLY APPROVED
SUSMP PLAN
REV: NOVEMBER 12, 2009
DATE: SEPT. 15, 2004
LEGAL DESCRIPTION:
PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.
PSOMAS
28470 Avenue Stanford, Suite 300
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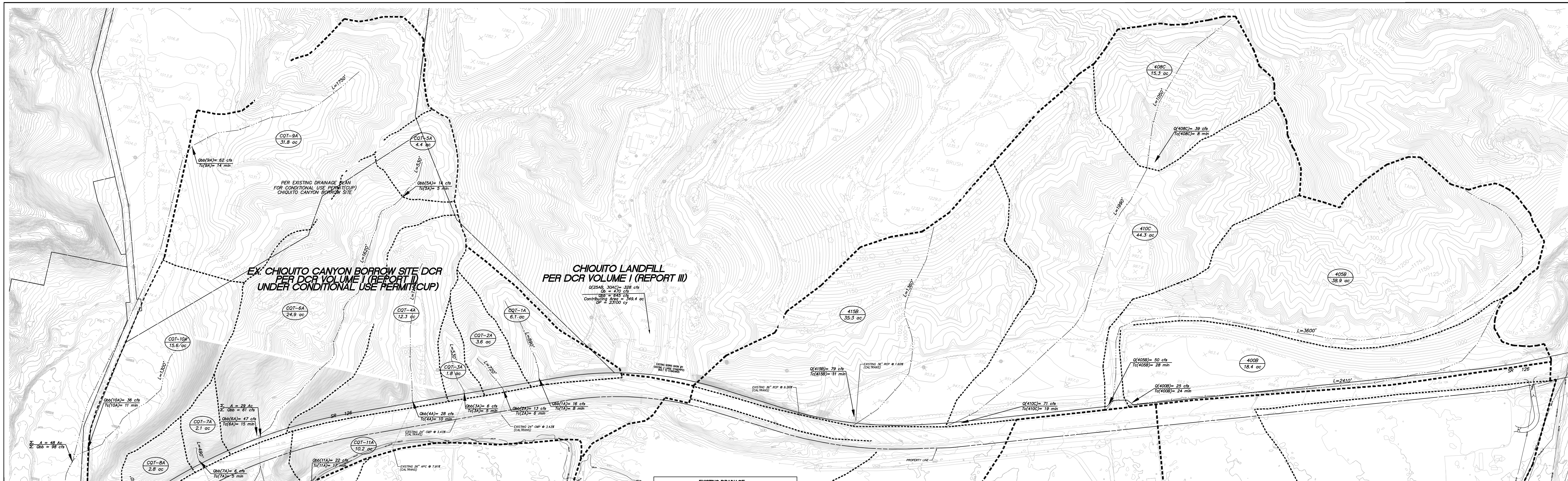
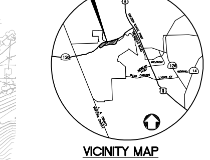
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO
DRAINAGE CONCEPT
VESTING TENTATIVE TRACT MAP
NO. 53108
NEWHALL RANCH - LANDMARK VILLAGE
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA
PROJECT NO. 1NRC0107.08
SHEET 3 OF 5
RV062-03





VICINITY MAP
NOT TO SCALE

LEGEND:

110A / 7.3 ac	SUB-BASIN NUMBER / SUB-BASIN AREA (ac)
	PROJECT BOUNDARY
	SUB-BASIN BOUNDARY
	DRAINAGE AREA BOUNDARY
	PROPERTY LINE
	EASEMENT
950	PROPOSED CONTOUR
	DAYLIGHT LINE
SD	STORM DRAIN
	EXISTING CMP INLET
	CATCH BASIN
X50. FP	EXISTING LACDWP Qcap FLOODPLAIN MAP (43-ML 23) PER DCR VOLUME V (PACE)
X50. FW	EXISTING LACDWP Qcap FLOODWAY MAP (43-ML 23) PER DCR VOLUME V (PACE)
REV. X50. FP	UPDATED EXISTING LACDWP Qcap FLOODPLAIN. PER DCR VOLUME V (PACE)
REV. X50. FW	UPDATED EXISTING LACDWP Qcap FLOODWAY. PER DCR VOLUME V (PACE)
Q50	Q-CAP (50yr-Proposed-burn&bulk) PER DCR VOLUME V
	BURIED BANK STABILIZATION
	DEBRIS BASIN CONTRIBUTING AREA
	FLOWPATH
	DEBRIS FLOWPATH
Q(sub)= 9 cfs	EXISTING SUBAREA Q(burned + bulked)
Tc(sub)= 15 min	EXISTING TIME OF CONCENTRATION
Q	Qunburned/develop
Qd	Qdeveloped
Qb	Qburned
Qbb	Qburned+bulked
Qbb	Qburned+bulked

DRAINAGE DESIGN CRITERIA:

1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 58%
   MULTI-FAMILY - 74%
   COMMERCIAL - 91%
   SCHOOL - 82%
   OPEN SPACE - 0%
   PARK - 15%
   ROADWAY - 91%
5. DPA= 09
6. BULK FACTOR= 1.27

EXISTING DRAINAGE

	INCREMENTAL SUB-BASINS					CUMULATIVE SUB-BASINS	
SUB-BASIN NUMBER	TIME OF CONCENTRATION (minutes)	SUB-BASIN AREA (acres)	Q (cfs)	Qb (cfs)	Qbb (cfs)	SUB-BASIN AREA (acres)	CUM. PEAK Qbb (cfs)
100A	22	32.7	30	41	52	---	---
110A	20	49.6	42	58	74	82.3	112
200A	17	17.3	17	23	29	---	---
210A	24	35.8	28	39	50	53.1	76
400B	24	18 4	14	20	25	---	---
405A	28	38.9	27	39	50	---	---
408C	8	15.3	24	31	39	---	---
410C	19	44.3	41	56	71	---	---
415B	11	35.3	47	62	79	---	---
420A	24	34.4	26	37	47	---	---
425A	20	39.9	35	48	61	226.5	304
500A	20	26.5	23	32	41	---	---
510A	24	40.0	31	44	53	66.5	81
620A	22	12 4	10	14	18	12 4	18
					SUBTOTAL	440.8	591
					CHIQUITO	127.3	185
					TOTAL	568.1	776

EXISTING CHIQUITO PROJECT DRAINAGE

		INCREMENTAL SUB-BASINS, 50-YEAR STORM EVENT				
SUB-BASIN NUMBER	TIME OF CONCENTRATION (minutes)	SUB-BASIN AREA (acres)	Qu (cfs)	Qb (cfs)	Qbb (cfs)	DEBRIS PRODUCTION (cy)
CQT-1A	8	6.1	10	13	16	165
CQT-2A	6	3.6	8	10	13	97
CQT-3A	5	1.8	4	5	6	49
CQT-4A	10	12.3	17	22	28	332
CQT-5A	5	4.4	9	11	14	119
CQT-6A	15	24.9	27	37	47	672
CQT-7A	5	2.1	4	5	6	57
CQT-8A	5	2.8	7	8	10	76
CQT-9A	14	31 8	36	49	62	859
CQT-10A	11	15.6	21	28	36	421
CQT-11A	17	10.2	16	17	22	54
CQT-12A	10	11.7	25	27	34	81
SUBTOTAL (PEAK FLOW)		127.3	107	146	185	2982

IS CONCEPTUALLY APPROVED

100' 50' 0' 100' 200'
1" 1/2" 0" 1" 2"
NOTE: For reduced size prints, original scale is in inches.

EXISTING AREAS OFFSITE

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

LEGAL DESCRIPTION:
PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
11444 West Olympic Boulevard, Suite 750
West Los Angeles, CA 90064-1549
(310) 954-3700 (310) 954-3777 (FAX)
ERIK WINATA R.C.E. NO. C67285 DATE

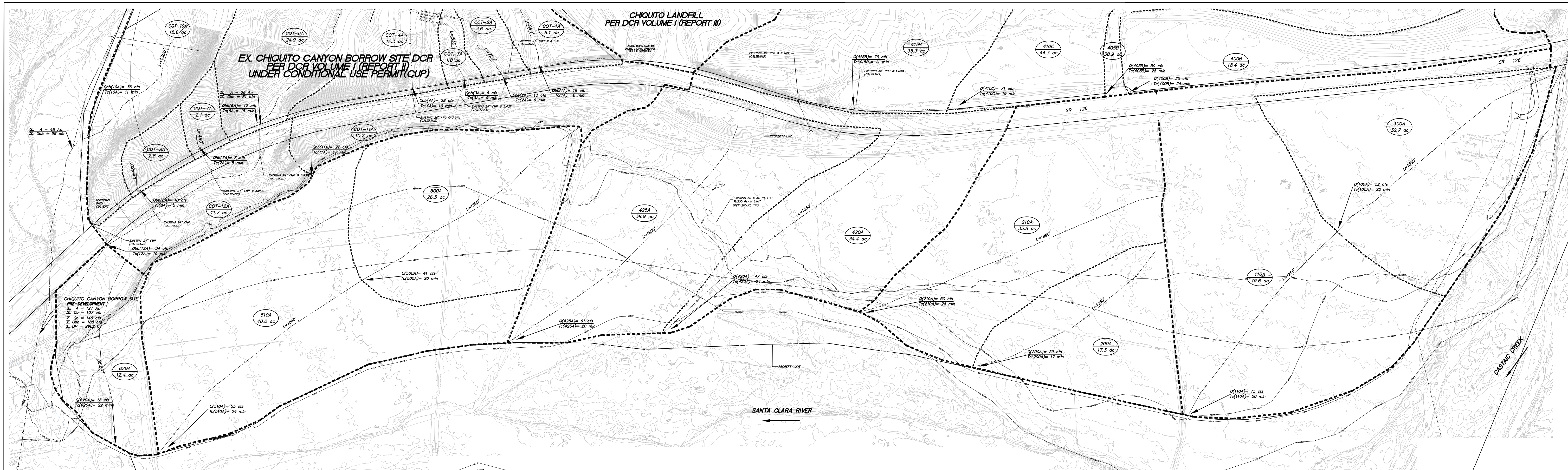
DESIGNED : NSK/EW/RB
DRAFTED : AC/ML
CHECKED : EW/RB

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. DAVID CROWDER

EXISTING DRAINAGE PLAN FOR VESTING TENTATIVE TRACT MAP NO. 53108
NEWHALL RANCH - LANDMARK VILLAGE
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 4 OF 5

RV062-04



LEGEND:

110A / 007.3 ac	SUB-BASIN NUMBER / SUB-BASIN AREA (ac)
Q(sub)= 9 cfs	EXISTING SUBAREA Q(burned + bulked)
Tc(sub)= 15 min	EXISTING TIME OF CONCENTRATION
Q	Qunburned/develop
Qd	Qdeveloped
Qb	Qburned
Qbb	Qburned+bulked
Qbb	Qburned+bulked

DRAINAGE DESIGN CRITERIA:

1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 58%, MULTI-FAMILY - 74%, COMMERCIAL - 91%, SCHOOL - 82%, OPEN SPACE - 0%, PARK - 15%, ROADWAY - 91%
5. DPA= 09
6. BULK FACTOR= 1.27

TOTAL PRE-DEVELOPMENT
A= 568 Ac
Qbb= 776 cfs

EXISTING DRAINAGE

	INCREMENTAL SUB-BASINS					CUMULATIVE SUB-BASINS	
SUB-BASIN NUMBER	TIME OF CONCENTRATION (minutes)	SUB-BASIN AREA (acres)	Q (cfs)	Qb (cfs)	Qbb (cfs)	SUB-BASIN AREA (acres)	Qbb (cfs)
100A	22	32.7	30	41	52	---	---
110A	20	49.6	42	58	74	82.3	112
200A	17	17.3	17	23	29	---	---
210A	24	35.8	28	39	50	53.1	76
400B	24	18 4	14	20	25	---	---
405A	28	38.9	27	39	50	---	---
408C	8	15.3	24	31	39	---	---
410C	19	44.3	41	56	71	---	---
415B	11	35.3	47	62	79	---	---
420A	24	34.4	26	37	47	---	---
425A	20	39.9	35	48	61	226.5	304
500A	20	26.5	23	32	41	---	---
510A	24	40.0	31	44	53	66.5	81
620A	22	12.4	10	14	18	12.4	18
					SUBTOTAL	440 8	591
					CHIQUITO	127.3	185
					TOTAL	568.1	776

EXISTING CHIQUITO PROJECT DRAINAGE

SUB-BASIN NUMBER	TIME OF CONCENTRATION (minutes)	INCREMENTAL SUB-BASINS, 50-YEAR STORM EVENT: SUB-BASIN AREA (acres)	Qu (cfs)	Qb (cfs)	Qbb (cfs)	DEBRIS PRODUCTION (cy)
CQT-1A	8	6.1	10	13	16	165
CQT-2A	6	3.6	8	10	13	97
CQT-3A	5	1.8	4	5	6	49
CQT-4A	10	12.3	17	22	28	332
CQT-5A	5	4.4	9	11	14	119
CQT-6A	15	24.9	27	37	47	672
CQT-7A	5	2.1	4	5	6	57
CQT-8A	5	2.8	7	8	10	76
CQT-9A	14	31.8	36	49	62	859
CQT-10A	11	15.6	21	28	36	421
CQT-11A	17	10.2	16	17	22	54
CQT-12A	10	11.7	25	27	34	81
SUBTOTAL		127.3	107	146	185	2982

NOTE: For reduced size prints, original scale is in inches.

IS CONCEPTUALLY APPROVED

EXISTING AREAS ON-SITE

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

LEGAL DESCRIPTION:
PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

PSOMAS
28470 Avenue Stanford, Suite 300
Santa Clarita, CA 91355
(661) 219-6000 (661) 775-2718 (FAX)
ERIK WINATA R.C.E. NO. C67285 DATE

DESIGNED : NSK/EW/JJ
DRAFTED : AC/ML
CHECKED : EW/RB

OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

EXISTING DRAINAGE PLAN FOR VESTING TENTATIVE TRACT MAP NO. 53108
NEWHALL RANCH - LANDMARK VILLAGE
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 5 OF 5

RV062-05

REPORT II

OFF-SITE BORROW AREAS
(UNDER CONDITIONAL USE PERMIT)
DRAINAGE CONCEPT REPORT

PSOMAS

OFF-SITE BORROW AREAS
(UNDER CONDITIONAL USE PERMIT)
REVISED DRAINAGE CONCEPT REPORT

November 2009

OFF-SITE BORROW AREAS
(UNDER CONDITIONAL USE PERMIT)
REVISED DRAINAGE CONCEPT REPORT

Psomas Project No: 1NRC010708
Prepared: November 2009

Prepared for:

NEWHALL LAND
23823 W. Valencia Blvd.
Valencia, CA 91355-2194
Telephone: (661) 255-4000

Prepared by:

PSOMAS
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Santa Clarita, California 91355
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11/12/09

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APPENDICES

1. **LACDPW Hydrology Data**
2. **Existing Condition TC Calculations**
3. **Proposed Condition TC Calculations**
4. **Existing Condition LACDPW MORA Calculations**
5. **Proposed Condition LACDPW MORA Calculations**
6. **Erosion and Sediment Control BMP Fact Sheets**
7. **Hydrology Exhibits for Chiquito Canyon Borrow Site**
8. **Hydrology Exhibits for Adobe Canyon Borrow Site**

1.0 Introduction

NOTE: This Drainage Report was conceptuall lopment Division, Los Angeles County Department of Public Work on July 5, 2007. A revision of this drainage report is needed for consistency with revised proposed grading on Adobe Canyon borrow site and Chiquito Canyon borrow site since approval on July 5, 2007.

Project Background

Psomas has been retained by the Newhall Land Co. to prepare a drainage concept report for the two off-site borrow areas associated with the Tentative Tract Map No. 53108 for the proposed Landmark Village development. The borrow areas are under a Conditional Use Permit (CUP), not part of the tentative tract map itself. The Chiquito Canyon borrow site is located north of Landmark Village, just north of SR 126. The Adobe Canyon borrow site is located south of Landmark Village, just south of the Santa Clara River.

Purpose and Scope

The project falls under the jurisdiction of the Los Angeles County Department of Public Works (LACDPW). The purpose of this drainage concept report is:

- To meet Los Angeles County Land Development requirements in support of the tentative tract map submittal, allowing final design and construction to proceed in a timely manner;
- To determine the proposed development's impact to existing hydrologic conditions;
- To provide sufficient detailed information to support detailed hydraulic design of storm drainage facilities; and
- To document that the Los Angeles County Standard Urban Stormwater Mitigation Plan (SUSMP) requirements will be met.

It should be noted that detailed storm drain and debris basin analyses is beyond the scope of this drainage concept.

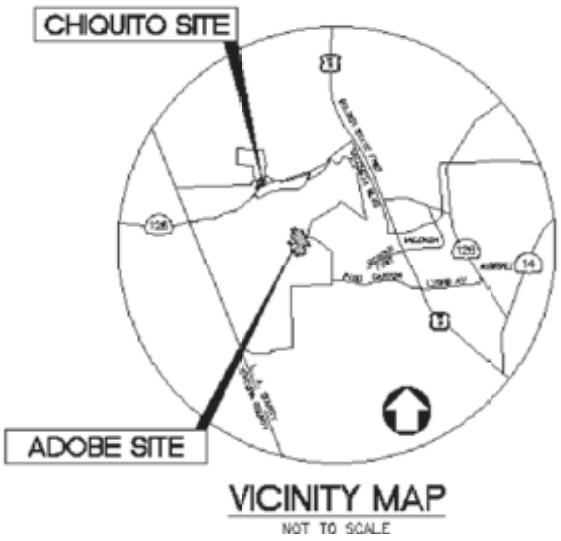


Figure 1. Location Map

2.0 Hydrology

General Approach

The watershed for each borrow site area was divided into several sub-basins that were identified and characterized for both existing and proposed conditions. Computer modeling was used to estimate the runoff for the 50-year storm events. The analysis considered with the additional bulking factors where appropriate to size sediment/debris facilities.

For the Chiquito Canyon site proposed condition, a combination of burned hydrology, and burned and bulked hydrology was considered for the discharges from the site, since sediment-trapping devices will be installed at the downstream end of the project for certain areas only. Debris basins are not planned for sub-basins CQT-9A and CQT-10A for the following reasons: the existing condition generates more debris than the proposed condition, the sub-basins drain directly to Chiquito Canyon, the improvements are temporary in nature,(i.e. a borrow site for constructing the proposed development) a -basins.

Data Sources

The primary sources of data were the *LACDPW Hydrology/ Sedimentation Manual and Appendices* (LACDPW 1991, 1992, 1993, 2002) and the LA County *Standard Urban Stormwater Mitigation Plan* (March 2000). Other key sources included the *TC (v1.0),* and *MORA (v7.2)* manuals and the Newhall Ranch Specific Plan Master Hydrology and Drainage Concept, Sikand December 1999 (appended for reference).

Watershed Characteristics

The Chiquito borrow site is bounded by SR 126 on the south, Chiquito Canyon on the west, and the Chiquito Landfill on the east. The total drainage area for the site encompasses approximately 73 acres and flows predominantly south to existing culverts under SR 126; a small portion drains west to Chiquito Canyon. The majority of the area is undeveloped land with steep to moderate slopes.

The Adobe borrow site is located south of the Santa Clara River and is bounded by Long Canyon to the west and the proposed Mesas East development to the east. The total drainage area for the site is about 214 acres and flows generally northwest and westerly. The majority of the land is undeveloped with steep to moderate slopes.

The soil of the two watersheds is classified by Los Angeles County to be Type 020 & 097, as shown on the figure (LACDPW 1-H1.43) included in Appendix 1. The figure also shows that the drainage areas of the two sites are associated with Debris Potential Area (DPA) numbers 08 and 09. As previously discussed, the proposed use of sediment trapping devices preclude the necessity of bulking factors in the proposed condition for the storm drain system leaving Chiquito borrow site. The peak bulking factor curves and peak debris production rates used in the analysis are included in Appendix 1 (LACDPW P2 & P5).

The Chiquito and Adobe borrow sites are located in Los Angeles County's Rainfall Isohyets 5.8, as shown on the aforementioned figure in Appendix 1.

The drainage area for the Chiquito site consists of twelve (12) sub-basins that independently drain toward the Santa Clara River, as shown on the Existing Condition Hydrology Map included with this report. The sub-basins drain southwesterly on site, ultimately tributary to the Santa Clara River.

The drainage area for the Adobe site consists of eight (8) sub-basins that independently drain toward the Santa Clara River, as shown on the Existing Condition Hydrology Map included with this report. The sub-basins drain northwesterly on site, ultimately tributary to the Santa Clara River.

The *LACDPW TC (TC_calc_depth.xls, June2002)* program was used to calculate the time of concentration for each sub area. The *LACDPW MORA (v7.2)* computer program was used for the *Modified Rational Method* calculations and calculation of peak runoff rates and hydrographs. The TC calculations and MORA program output are provided in Appendix 2-3 and 4-5, respectively. In accordance with LACDPW requirements, the 50-year burned-a -bulked storm event was used as the main design storm in the analysis.

In the tables that follow, the following abbreviations apply:

Q_u = Runoff, Unburned and Unbulked
Q_b = Burned Runoff

Q_{bb} = Burned and Bulked Runoff
Q_{b+d} = Burned and Developed Runoff
Q_{bb+d} = Burned and Bulked and Developed Runoff
Q_{design} = Runoff Value Used for Design

Existing Condition Results

Existing condition hydrology results for the Capital Flood storm event are summarized in Tables 1a and 1b below.

Table 1a: Existing Condition Hydrology Summary – Chiquito Site

Sub basins	Area (Ac)	50-Year Storm Event			
		Time of Conc. (min)	Q_u (cfs)	Q_b (cfs)	Q_{bb} (cfs)
CQT-1A	6.1	8	10	13	16
CQT-2A	3.6	6	8	10	13
CQT-3A	1.8	5	4	5	6
CQT-4A	12.3	10	17	22	28
CQT-5A	4.4	5	9	11	14
CQT-6A	24.9	15	27	37	47
CQT-7A	2.1	5	4	5	6
CQT-8A	2.8	5	7	8	10
CQT-9A	31.8	14	36	49	62
CQT-10A	15.6	11	21	28	36
CQT-11A	10.2	17	16	17	22
CQT-12A	11.7	10	25	27	34
Total	**127.3**		107	146	**185**

Table 1b: Existing Condition Hydrology Summary – Adobe Site

Sub basins	Area (Ac)	50-Year Storm Event			
		Time of Conc. (min)	Q_u (cfs)	Q_b (cfs)	$Q_{bb`}$ (cfs)
ADB-1A	35.8	9	67	77	105
ADB-2A	40.0	10	71	82	112
ADB-3A	24.0	11	40	47	64
ADB-4B	16.7	11	28	32	44
ADB-5B	39.9	16	53	63	86
ADB-7C	27.4	12	53	51	69
ADB-8C	12.9	10	21	26	36
ADB-9C	16.6	8	33	38	52
Total	**213.3**		359	416	**568**

Post-Project Condition Results

The post-project condition hydrologic analysis was based on the preliminary concept drainage plan prepared by Psomas, included with this report. The post-project (developed) Condition Drainage Concept Map is also included with this report. Post-development drainage patterns do not significantly differ from the aforementioned existing characteristics.

As with the existing condition analysis, the *LACDPW TC* program was used to calculate the time of concentration, and the *LACDPW MORA* computer program was used to calculate the peak runoff rates and hydrographs. The calculations and program output are provided in Appendix 3 and 5 respectively. In accordance with the *LACDPW Hydrology/Sedimentation Appendix*, imperviousness coefficients are as follows:

Table 2: Land Use Impervious Values

Land Use	Percent Imperviousness
Open Space/Natural	0%
Borrow Site Grading	0%

Post-project condition hydrology results for the 50-year storm event are summarized in Table 3a and 3b.

Table 3a: Post-Project Condition Hydrology Summary – Chiquito Site

Sub basins	Area (Ac)	50-Year Storm Event				
		Time of Conc. (min)	Q_u (cfs)	Q_{b+d} (cfs)	Q_{bb+d} (cfs)	Q_{design} [A] (cfs)
CQT-1A	29.9	11	42	54	69	69
CQT-2A	6.4	6	12	15	19	19
CQT-3B	4.4	5	9	11	14	11
CQT-4B	19.6	14	24	31	39	31
CQT-5B	6.94	7	13	16	20	16
CQT-6B	5.9	7	11	14	18	14
CQT-7B	13.3	9	24	28	35	28
CQT-9C	7.7	8	13	17	22	17
CQT-10C	8.8	11	12	16	20	16
CQT-11C	11.8	18	20	-	-	20
CQT-12C	2.2	16	2	-	-	2
CQT-13C	7.5	22	11	-	-	11
Total	**124.4**		109	159	202	**159**[A]

Notes

A. Burned flows for Subareas CQT-3B, CQT-4B, CQT-5B, CQT-7B, CQT-9C Burned and bulked flows for Subareas CQT-1A and, CQT-2A, Developed flows for Subareas CQT-11C , CQT-12C and CQT-13C

Table 3b: Post-Project Condition Hydrology Summary – Adobe Site

Sub basins	Area (Ac)	50-Year Storm Event				
		Time of Conc. (min	Q_u (cfs)	Q_{b+d} (cfs)	Q_{bb+d} (cfs)	Q_{design}[A](cfs)
ADB-1A	25.2	9	47	54	74	54
ADB-2A	46.8	9	87	92	103	92
ADB-3A	25.5	10	45	49	58	49
ADB-4A	23.5	11	39	39	39	39
ADB-5B	14.24	8	28	33	44	44
ADB-6B	40.7	21	46	53	69	69
ADB-7C	17	10	30	35	47	47
ADB-8C	6.6	5	17	19	26	26
ADB-9C	15.1	6	36	38	46	46
Total	**214.6**		**376**	**409**	**498**	**466**

Notes

A. Burned and Developed flow for Sub-basins 1A thru 4A, plus Burned and Bulked and Developed flow for the remaining Sub-basins.

Baseline Hydrology Comparison

A comparison of existing and proposed peak flow rates are provided as Table 4a and 4b. In accordance with Los Angeles County hydrology method, existing condition burned and bulked flow is assessed with proposed condition flow burned, unbulked due to the proposed addition of debris facilities where needed.

Table 4a: Existing vs. Proposed Condition Hydrology Comparison Summary – Chiquito Site

50-Year Storm Event						
Sub area	Area (Ac)			Q_{50} (cfs)		
	Exist.[A]	Prop.[B]	Diff.	Exist[C].	Prop.[D]	Diff.
Total Study Area	**127**	**124**	**-3**	**185**	**159**	**-26**

Notes

A. Drainage tributary area includes portion of Newhall Ranch – Landmark Village Tract 53108.

B. Drainage tributary area reduced because a portion of the area is accounted for in the on-site hydrology study for Newhall Ranch - Landmark Village Tract 53108.

C. Burned and bulked flow

D. Burned and Developed flow for Sub-basins 1/4A,5A,6A, plus Burned and Bulked and Developed flow for Sub-basins 9A,10A, plus Developed flow for Sub-basins 7/8A,11A,12A

Table 4b: Existing vs. Proposed Condition Hydrology Comparison Summary – Adobe Site

50-Year Storm Event						
Sub area	Area (Ac)			Q_{50} (cfs)		
	Exist.	Prop.	Diff.	Exist[A].	Prop.[B]	Diff.
Total Study Area	213.3	214.6	+1.3	568	466	-102

Notes

A. Burned and bulked flow

B. Burned and Developed flow for Sub-basins 1A thru 4A, plus Burned and Bulked and Developed flow for the remaining Sub-basins.

3.0 Standard Urban Stormwater Mitigation Plan (SUSMP)

Instead of volume- or flow-based Treatment Control BMPs, the Los Angeles County Standard Urban Stormwater Mitigation Plan (SUSMP) requirements will be met by Erosion and Sediment Control BMPs, which decrease the potential of slopes and channels to erode and impact stormwater runoff. LA County SUSMP guidelines specify BMPs such as the following:

- Conveying runoff safely from the tops of slopes and stabilizing disturbed slopes
- Utilizing natural drainage systems to the maximum extent practicable (MEP)
- Stabilizing permanent channel crossings
- Vegetating slopes with native or drought tolerant vegetation
- Installing energy dissipators, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion

As shown on the Drainage Concept Exhibits, both temporary and permanent Erosion and Sediment Control BMPs have been incorporated into the two sites. Temporary BMPs will be implemented during the 'construction phase' of the site, while permanent BMPs will be implemented upon completion of construction. Potential erosion and sediment control BMP options are summarized in the following table; however, BMPs will be finalized during the completion of the Construction Stormwater Pollution Prevention Plan (SWPPP).

Table 5: Potential Temporary and Permanent Erosion/Sediment Control BMP Options

BMP	Fact Sheet # [A]	Temporary	Permanent
Scheduling	EC-1	X	
Hydraulic Mulch [B]	EC-3	X	X
Hydroseeding [B]	EC-4	X	X
Soil Binders	EC-5	X	
Geotextiles and Mats	EC-7	X	
Earth Dikes and Drainage Swales	EC-9	X	X
Velocity Dissipation Devices	EC-10	X	X
Slope Drains	EC-11	X	X
Silt Fence	SE-1	X	
Desilting (Sediment) Basin	SE-2	X	X
Sediment Trap	SE-3	X	
Check Dams	SE-4	X	X
Gravel Bag Berm	SE-6	X	
Street Sweeping and Vacuuming	SE-7	X	
Sandbag Barrier	SE-8	X	
Straw Bale Barrier	SE-9	X	
Storm Drain Inlet Protection	SE-10	X	
Wind Erosion Control	WE-1	X	

Stabilized Construction Entrance/Exit	TC-1	X	
Stabilized Construction Roadway	TC-2	X	
Entrance/Outlet Tire Wash	TC-3	X	

A. CASQA, 2003.
B. Requires BMP monitoring plan until vegetation is permanently established.

BMP fact sheets for the above Sediment and Erosion Control BMPs are included in Appendix 6. Final selection and detailed design of these BMPs is beyond the scope of this report. However, the report and exhibits show that erosion and sediment control SUSMP requirements can be met.

4.0 Limitations

This report was prepared to comply with the guidelines established by the County of Los Angeles and their representatives. Evaluation of the appropriateness of these guidelines and the accuracy of the County data were beyond the scope of this work.

Usage of the report is limited to address the purpose and scope previously defined by the project owner, Newhall Land. Psomas shall not be held responsible for any unauthorized application of this report and the contents herein.

The opinions represented in this report have been derived in accordance with current standards of civil engineering practice. No other warranty is expressed or implied.

5.0 References

California Stormwater Quality Association, *Stormwater Best Management Practice Handbook – Construction* (January 2003)

Los Angeles County Department of Public Works, *LACDPW Hydrology/ S*
)

Los Angeles County Department of Public Works, *LACDPW TC v1.0 Manual, TC_calc_depth.xls* (December 1991, June 2002)

Los Angeles County Department of Public Works, *LACDPW Modified Rational Method, MORA (MORA) v7.2 Manual* (September 2002)

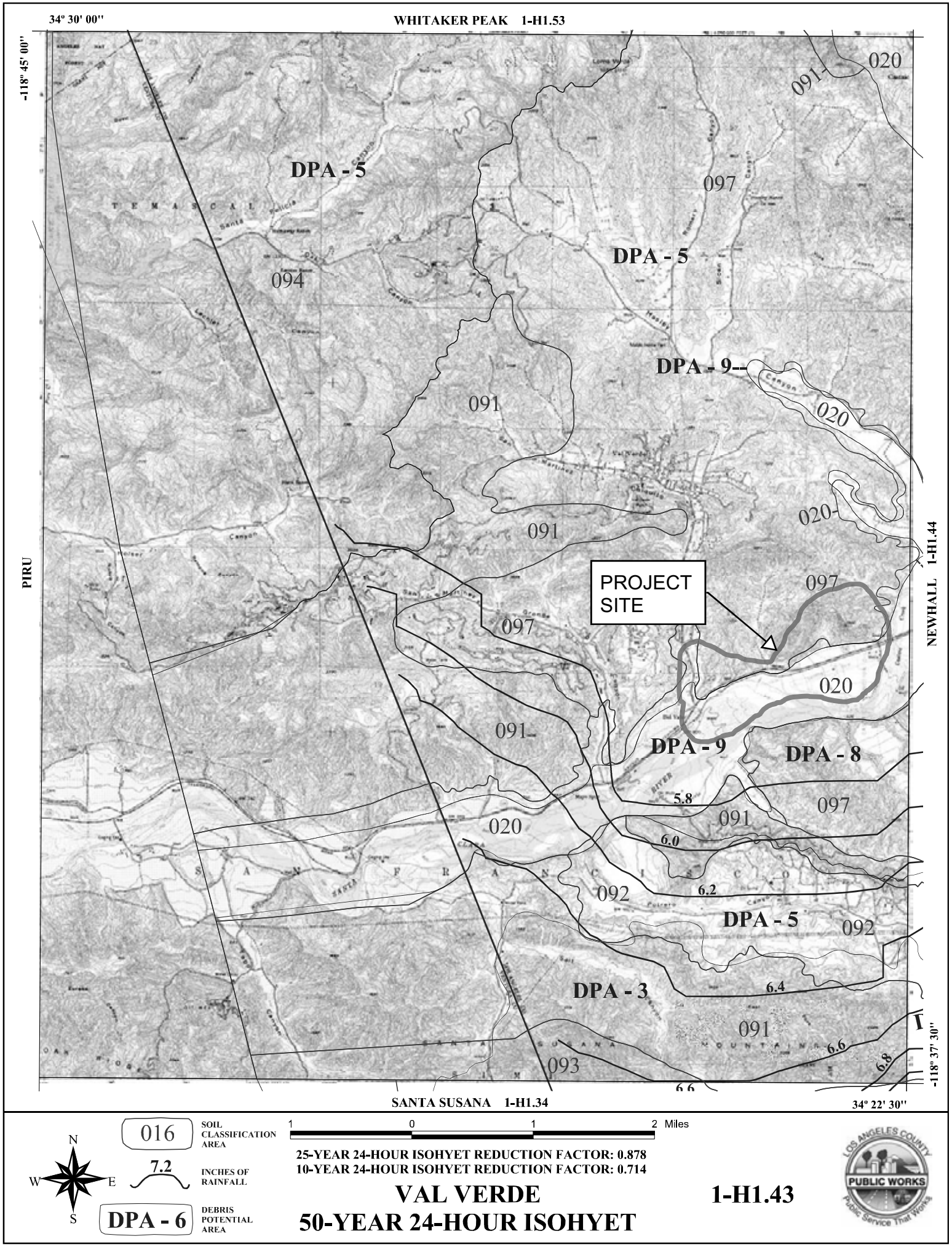
Los Angeles Regional Water Quality Control Board, *Standard Urban Stormwater Mitigation Plan for Los Angeles County and Cities in Los Angeles County* (March 2000)

Newhall Ranch, Newhall Ranch Specific Plan Master Hydrology and Drainage Concept (Sikand, December, 1999)

APPENDICES

APPENDIX 1

LACDPW HYDROLOGY DATA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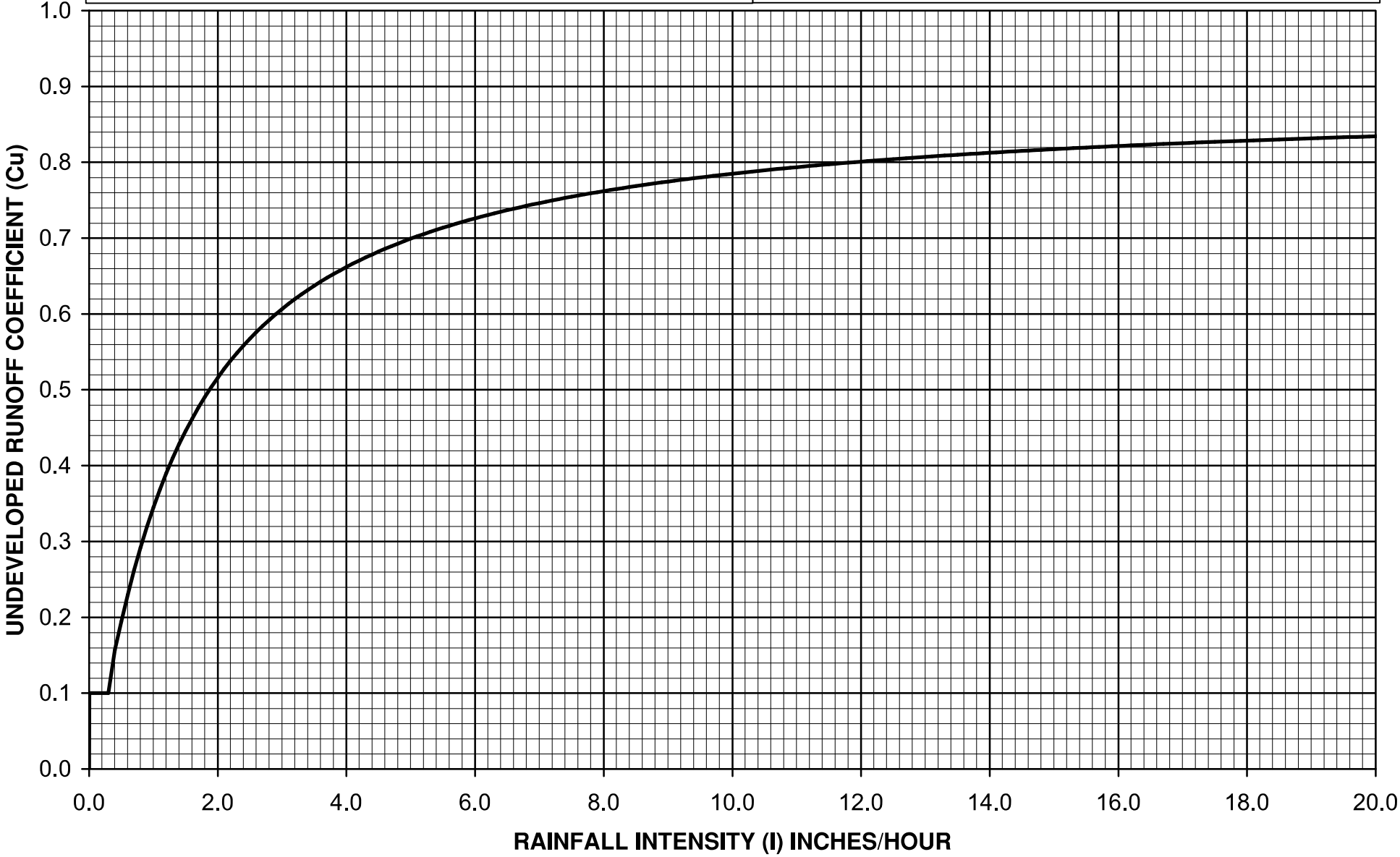
-118° 45' 00"
PIRU
NEWHALL 1-H1.44
-118° 37' 30"
SANTA SUSANA 1-H1.34
34° 22' 30"
DPA - 5
DPA - 5
DPA - 9
DPA - 9
DPA - 8
DPA - 5
DPA - 3
PROJECT SITE
091
020
097
094
092
093
5.8
6.0
6.2
6.4
6.6
6.8
6.6
SANTA SUSANA MOUNTAINS
SAN FERNANDO
016
SOIL CLASSIFICATION AREA
7.2
INCHES OF RAINFALL
DPA - 6
DEBRIS POTENTIAL AREA
N
W
E
S
1 0 1 2 Miles
25-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.878
10-YEAR 24-HOUR ISOHYET REDUCTION FACTOR: 0.714
VAL VERDE
50-YEAR 24-HOUR ISOHYET
1-H1.43
LOS ANGELES COUNTY
PUBLIC WORKS
Public Service That Works

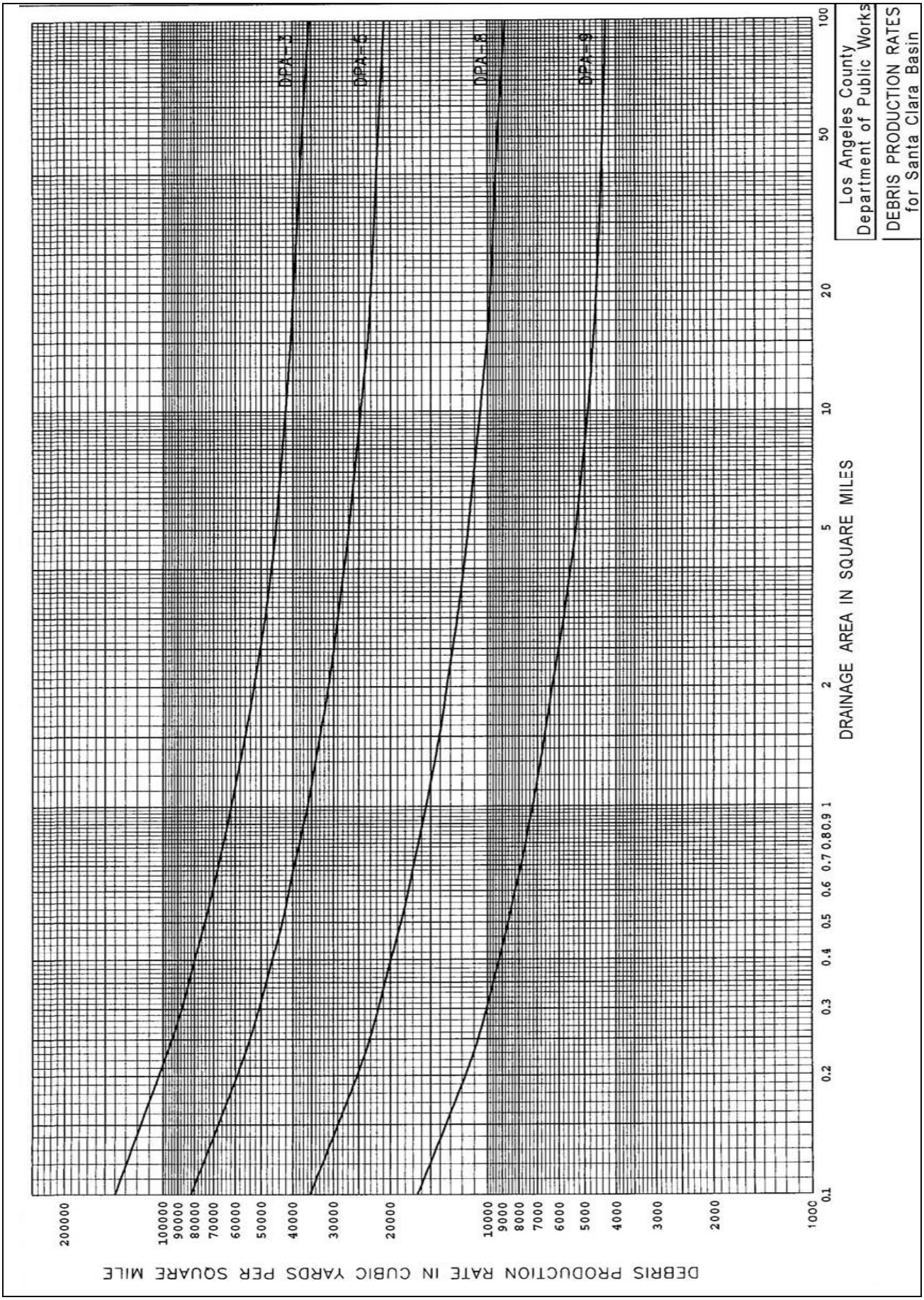
$C_D = (0.9 * IMP) + (1.0 - IMP) * C_U$

Where: C_D = Developed Runoff Coefficient
IMP = Proportion Impervious
C_U = Undeveloped runoff coefficient

Los Angeles County Department of Public Works

RUNOFF COEFFICIENT CURVE
SOIL TYPE NO. 020




DEBRIS PRODUCTION RATE IN CUBIC YARDS PER SQUARE MILE
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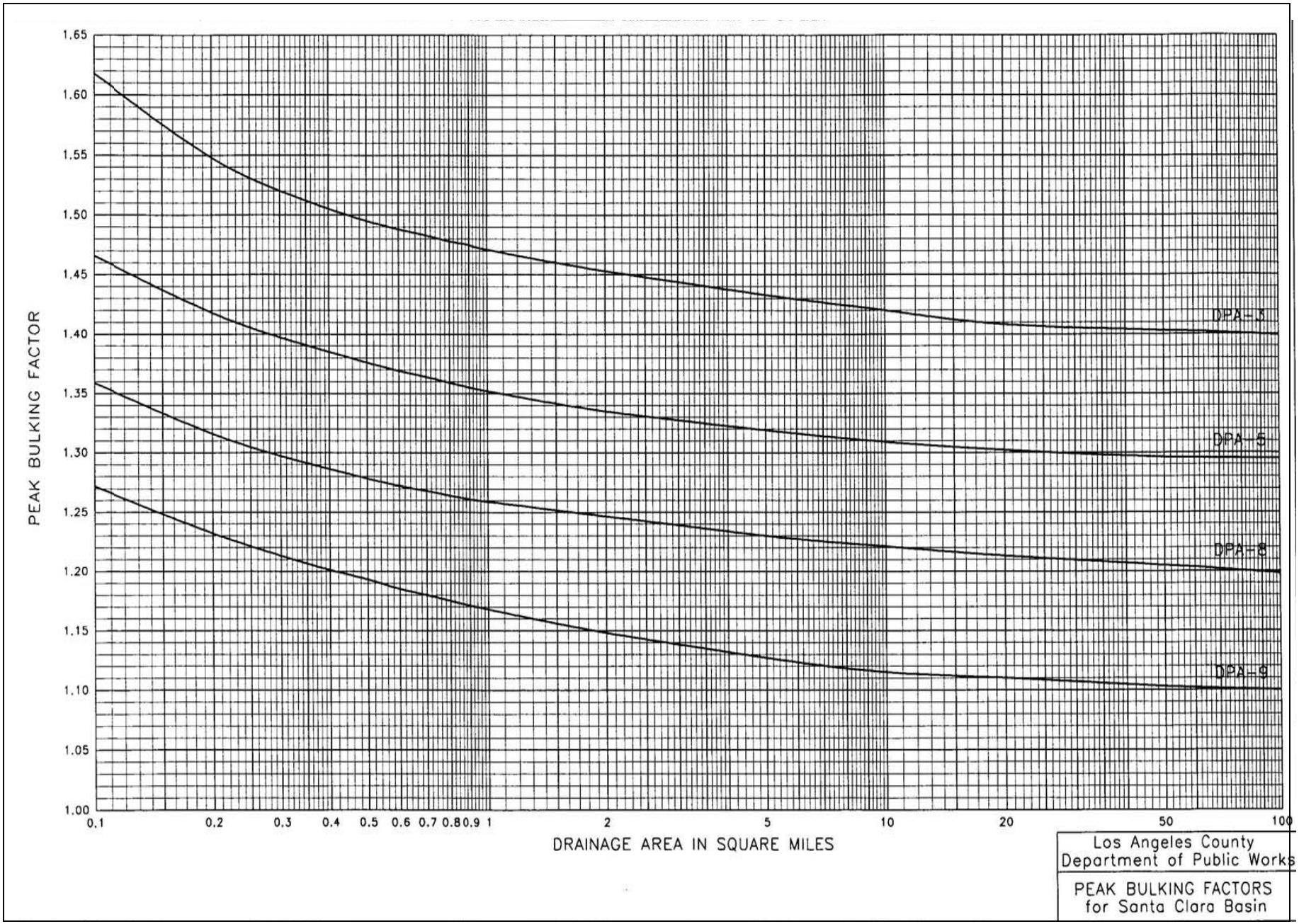
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
DPA-1
DPA-2
DPA-3
DPA-4
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
5
10
20
50
100
DRAINAGE AREA IN SQUARE MILES
Los Angeles County
Department of Public Works
DEBRIS PRODUCTION RATES
for Santa Clara Basin


PEAK BULKING FACTOR
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1.60
1.55
1.50
1.45
1.40
1.35
1.30
1.25
1.20
1.15
1.10
1.05
1.00
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
2
5
10
20
50
100
DRAINAGE AREA IN SQUARE MILES
DPA-3
DPA-5
DPA-8
DPA-9
Los Angeles County
Department of Public Works
PEAK BULKING FACTORS
for Santa Clara Basin

APPENDIX 2

TC CALCULATIONS:
EXISTING CONDITION

Chiquito Borrow Site
Existing Condition
TC Data File

Project	Subarea	Area	%imp	Frequency	Soil Type	Length	Slope	Isohyet
Area (E)	1A	35.8	0	2	20	1755	0.614	2.2
Area (E)	2A	40	0	2	20	1035	0.059	2.2
Area (E)	3A	24	0	2	20	1095	0.046	2.2
Area (E)	1B	16.7	0	2	20	1560	0.138	2.2
Area (E)	2B	39.9	0	2	20	2490	0.092	2.2
Area (E)	1C	27.4	0	2	20	1985	0.217	2.2
Area (E)	1D	12.9	0	2	20	1450	0.221	2.2
Area (E)	1E	16.6	0	2	20	1110	0.270	2.2
Area (E)	1A	35.8	0	5	20	1755	0.614	3.4
Area (E)	2A	40	0	5	20	1035	0.059	3.4
Area (E)	3A	24	0	5	20	1095	0.046	3.4
Area (E)	1B	16.7	0	5	20	1560	0.138	3.4
Area (E)	2B	39.9	0	5	20	2490	0.092	3.4
Area (E)	1C	27.4	0	5	20	1985	0.217	3.4
Area (E)	1D	12.9	0	5	20	1450	0.221	3.4
Area (E)	1E	16.6	0	5	20	1110	0.270	3.4
Area (E)	1A	35.8	0	10	20	1755	0.614	4.1
Area (E)	2A	40	0	10	20	1035	0.059	4.1
Area (E)	3A	24	0	10	20	1095	0.046	4.1
Area (E)	1B	16.7	0	10	20	1560	0.138	4.1
Area (E)	2B	39.9	0	10	20	2490	0.092	4.1
Area (E)	1C	27.4	0	10	20	1985	0.217	4.1
Area (E)	1D	12.9	0	10	20	1450	0.221	4.1
Area (E)	1E	16.6	0	10	20	1110	0.270	4.1
Area (E)	1A	35.8	0	25	20	1755	0.614	5.1
Area (E)	2A	40	0	25	20	1035	0.059	5.1
Area (E)	3A	24	0	25	20	1095	0.046	5.1
Area (E)	1B	16.7	0	25	20	1560	0.138	5.1
Area (E)	2B	39.9	0	25	20	2490	0.092	5.1
Area (E)	1C	27.4	0	25	20	1985	0.217	5.1
Area (E)	1D	12.9	0	25	20	1450	0.221	5.1
Area (E)	1E	16.6	0	25	20	1110	0.270	5.1
Area (E)	1A	35.8	0	50	20	1755	0.614	5.8
Area (E)	2A	40	0	50	20	1035	0.059	5.8
Area (E)	3A	24	0	50	20	1095	0.046	5.8
Area (E)	1B	16.7	0	50	20	1560	0.138	5.8
Area (E)	2B	39.9	0	50	20	2490	0.092	5.8
Area (E)	1C	27.4	0	50	20	1985	0.217	5.8
Area (E)	1D	12.9	0	50	20	1450	0.221	5.8
Area (E)	1E	16.6	0	50	20	1110	0.270	5.8
Area (E)	1A	35.8	0	100	20	1755	0.614	6.5
Area (E)	2A	40	0	100	20	1035	0.059	6.5
Area (E)	3A	24	0	100	20	1095	0.046	6.5
Area (E)	1B	16.7	0	100	20	1560	0.138	6.5
Area (E)	2B	39.9	0	100	20	2490	0.092	6.5
Area (E)	1C	27.4	0	100	20	1985	0.217	6.5
Area (E)	1D	12.9	0	100	20	1450	0.221	6.5
Area (E)	1E	16.6	0	100	20	1110	0.270	6.5
Area (E)	1A	35.8	0	500	20	1755	0.614	8.2
Area (E)	2A	40	0	500	20	1035	0.059	8.2
Area (E)	3A	24	0	500	20	1095	0.046	8.2
Area (E)	1B	16.7	0	500	20	1560	0.138	8.2
Area (E)	2B	39.9	0	500	20	2490	0.092	8.2
Area (E)	1C	27.4	0	500	20	1985	0.217	8.2
Area (E)	1D	12.9	0	500	20	1450	0.221	8.2
Area (E)	1E	16.6	0	500	20	1110	0.270	8.2

Chiquito Borrow Site
Existing Condition
TC Calculation Results

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft)	Isohyet (in.	Tc-calculat	Intensity (ir	Cu	Cd	Flowrate (c	Tc Equation
Area (E)	1A	35.8	0	2	20	1755	0.614	2.2	30	0.57	0.22	0.22	4.49	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	2	20	1035	0.059	2.2	30	0.57	0.22	0.22	5.02	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	2	20	1095	0.046	2.2	30	0.57	0.22	0.22	3.01	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	2	20	1560	0.138	2.2	30	0.57	0.22	0.22	2.09	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	2	20	2490	0.092	2.2	30	0.57	0.22	0.22	5	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	2	20	1985	0.217	2.2	30	0.57	0.22	0.22	3.44	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	2	20	1450	0.221	2.2	30	0.57	0.22	0.22	1.62	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	2	20	1110	0.27	2.2	30	0.57	0.22	0.22	2.08	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	5	20	1755	0.614	3.4	20	1.06	0.36	0.36	13.66	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	5	20	1035	0.059	3.4	22	1.01	0.35	0.35	14.14	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	5	20	1095	0.046	3.4	25	0.95	0.33	0.33	7.52	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	5	20	1560	0.138	3.4	26	0.93	0.32	0.32	4.97	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	5	20	2490	0.092	3.4	30	0.87	0.31	0.31	10.76	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	5	20	1985	0.217	3.4	29	0.89	0.31	0.31	7.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	5	20	1450	0.221	3.4	22	1.01	0.35	0.35	4.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	5	20	1110	0.27	3.4	17	1.14	0.37	0.37	7	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	10	20	1755	0.614	4.1	15	1.46	0.44	0.44	23	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	10	20	1035	0.059	4.1	17	1.38	0.42	0.42	23.18	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	10	20	1095	0.046	4.1	19	1.31	0.41	0.41	12.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	10	20	1560	0.138	4.1	20	1.28	0.4	0.4	8.55	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	10	20	2490	0.092	4.1	30	1.05	0.36	0.36	15.08	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	10	20	1985	0.217	4.1	22	1.22	0.39	0.39	13.04	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	10	20	1450	0.221	4.1	17	1.38	0.42	0.42	7.48	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	10	20	1110	0.27	4.1	13	1.56	0.45	0.45	11.65	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	25	20	1755	0.614	5.1	12	2.02	0.52	0.52	37.6	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	25	20	1035	0.059	5.1	13	1.94	0.51	0.51	39.58	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	25	20	1095	0.046	5.1	14	1.88	0.5	0.5	22.56	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	25	20	1560	0.138	5.1	15	1.82	0.49	0.49	14.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	25	20	2490	0.092	5.1	23	1.49	0.44	0.44	26.16	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	25	20	1985	0.217	5.1	16	1.76	0.48	0.48	23.15	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	25	20	1450	0.221	5.1	13	1.94	0.51	0.51	12.76	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	25	20	1110	0.27	5.1	10	2.2	0.53	0.53	19.36	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	50	20	1755	0.614	5.8	11	2.39	0.55	0.55	47.06	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	50	20	1035	0.059	5.8	12	2.29	0.54	0.54	49.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	50	20	1095	0.046	5.8	12	2.29	0.54	0.54	29.68	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	50	20	1560	0.138	5.8	13	2.21	0.54	0.54	19.93	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	50	20	2490	0.092	5.8	20	1.8	0.49	0.49	35.19	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	50	20	1985	0.217	5.8	14	2.13	0.53	0.53	30.93	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	50	20	1450	0.221	5.8	11	2.39	0.55	0.55	16.96	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	50	20	1110	0.27	5.8	9	2.63	0.57	0.57	24.89	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	100	20	1755	0.614	6.5	9	2.94	0.6	0.6	63.15	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	100	20	1035	0.059	6.5	10	2.8	0.59	0.59	66.08	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	100	20	1095	0.046	6.5	11	2.68	0.58	0.58	37.31	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	100	20	1560	0.138	6.5	12	2.57	0.57	0.57	24.46	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	100	20	2490	0.092	6.5	18	2.12	0.53	0.53	44.83	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	100	20	1985	0.217	6.5	12	2.57	0.57	0.57	40.14	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	100	20	1450	0.221	6.5	10	2.8	0.59	0.59	21.31	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	100	20	1110	0.27	6.5	8	3.11	0.61	0.61	31.49	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1A	35.8	0	500	20	1755	0.614	8.2	7	4.18	0.67	0.67	100.26	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2A	40	0	500	20	1035	0.059	8.2	8	3.92	0.66	0.66	103.49	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	3A	24	0	500	20	1095	0.046	8.2	9	3.71	0.65	0.65	57.88	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1B	16.7	0	500	20	1560	0.138	8.2	9	3.71	0.65	0.65	40.27	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	2B	39.9	0	500	20	2490	0.092	8.2	13	3.12	0.61	0.61	75.94	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1C	27.4	0	500	20	1985	0.217	8.2	10	3.53	0.64	0.64	61.9	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1D	12.9	0	500	20	1450	0.221	8.2	8	3.92	0.66	0.66	33.37	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
Area (E)	1E	16.6	0	500	20	1110	0.27	8.2	6	4.49	0.68	0.68	50.68	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Landmark Village - Newhall Ranch
Chiquito Canyon
Existing Drainage Condition

Subarea	Area (Ac)	Calcs Tc (min)	I_{50} (in/hr)	Cu	Cd	Q_{50} (cfs)	Area Undeveloped (Ac)	K (1)	Q_{50b} (cfs) (2)	Q_{50bb} (cfs) (3)	DP (cy) (4)
CQT-1A	6.10	8	2.77	0.59	0.59	9.97	6.1	0.61	12.67	16.09	165
CQT-2A	3.60	6	3.18	0.62	0.62	7.10	3.6	0.60	8.83	11.22	97
CQT-3A	1.80	5	3.46	0.63	0.63	3.92	1.8	0.60	4.85	6.16	49
CQT-4A	12.30	10	2.50	0.56	0.56	17.22	12.3	0.62	22.41	28.46	332
CQT-5A	4.40	5	3.46	0.63	0.63	9.59	4.4	0.60	11.86	15.07	119
CQT-6A	24.90	15	2.06	0.52	0.52	26.67	24.9	0.63	35.81	45.48	672
CQT-7A	2.10	5	3.46	0.63	0.63	4.58	2.1	0.60	5.66	7.19	57
CQT-8A	2.80	5	3.46	0.63	0.63	6.10	2.8	0.60	7.55	9.59	76
CQT-9A	31.80	14	2.13	0.53	0.53	35.90	31.8	0.63	47.78	60.68	859
CQT-10A	15.60	11	2.39	0.55	0.55	20.51	15.6	0.62	26.89	34.15	421
CQT-11A	10.20	17	1.95	0.51	0.90	17.90	2.0	0.63	17.90	17.90	54
CQT-12A	11.70	10	2.50	0.56	0.90	26.33	3.0	0.62	26.33	26.33	81
Total	**127.3**					**107.0**			**146.0**	**185.0**	**2981**

(1) - K = Burn Factor from Appendix G-2, K = 0.677*(I ^-0.102)
(2) - Qb = A*I*(1-K)+K*Qu
(3) - Qbb = 1.27*Qb
(4) - DP = A*27 cy (Zone 9)

V:\Newhall\PRODDATA\53108-MP\ENGR\DOCS\Drainage Concept Reports\Borrow Site Hydrology Report\Chiquito Canyon Borrow Site\[CQT_HYDRO_EXIST.xls]Qbb

Adobe Borrow Site
EX Condition
TC Data File

Project	Subarea	Area	%imp	Frequency	Soil Type	Length	Slope	Isohyet
ADB	1A	35.8	0	50	97	1755	0.614	5.8
ADB	2A	40	0	50	97	1035	0.059	5.8
ADB	3A	24	0	50	97	1095	0.046	5.8
ADB	4B	16.7	0	50	97	1560	0.138	5.8
ADB	5B	39.9	0	50	97	2490	0.092	5.8
ADB	6C	27.4	0	50	97	1985	0.217	5.8
ADB	7C	12.9	0	50	97	1450	0.221	5.8
ADB	8C	16.6	0	50	97	1110	0.270	5.8

Landmark Village
EX Condition
TC Calculation Results

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft	Isohyet (in.)	Tc-calculated (min.)	Intensity (in./hr)	Cu	Cd	Flowrate (c	Tc Equation
ADB	1A	35.8	0	50	97	1755	0.614	5.8	9	2.63	0.71	0.71	66.85	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	2A	40	0	50	97	1035	0.059	5.8	10	2.5	0.71	0.71	71	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	3A	24	0	50	97	1095	0.046	5.8	11	2.39	0.7	0.7	40.15	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	4B	16.7	0	50	97	1560	0.138	5.8	11	2.39	0.7	0.7	27.94	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	5B	39.9	0	50	97	2490	0.092	5.8	16	2	0.67	0.67	53.47	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	6C	27.4	0	50	97	1985	0.217	5.8	12	2.29	0.69	0.69	43.29	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	7C	12.9	0	50	97	1450	0.221	5.8	10	2.5	0.71	0.71	22.9	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	8C	16.6	0	50	97	1110	0.27	5.8	8	2.77	0.72	0.72	33.11	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Landmark Village - Newhall Ranch
Adobe Canyon
Existing Drainage Condition

Subarea	Area (Ac)	Calcs Tc (min)	I_{50} (in/hr)	Cu	Cd	Q_{50} (cfs)	Area Undeveloped (Ac)	K (1)	Q_{50b} (cfs) (2)	Q_{50bb} (cfs) (3)	DP (cy) (4)
1A	35.8	9	2.63	0.71	0.71	66.85	35.8	0.61	77	105	1933
2A	40.0	10	2.50	0.71	0.71	71.00	40.0	0.62	82	112	2160
3A	24.0	11	2.39	0.70	0.70	40.15	24.0	0.62	47	64	1296
4B	16.7	11	2.39	0.70	0.70	27.94	16.7	0.62	32	44	902
5B	39.9	16	2.00	0.67	0.67	53.47	39.9	0.63	63	86	2155
7C	27.4	12	2.29	0.69	0.69	43.29	27.4	0.62	51	69	1480
8C	12.9	10	2.50	0.71	0.71	22.90	12.9	0.62	26	36	697
9C	16.6	8	2.77	0.72	0.72	33.11	16.6	0.61	38	52	896
Total	**213.3**					**359**			**417**	**567**	**11518**

(1) - K = Burn Factor from Appendix G-2, K = 0.677*(I ^-0.102)
(2) - Qb = A*I*(1-K)+K*Qu
(3) - Qbb = 1.36*Qb
(4) - DP = A * 54 cy (Zone 8)

APPENDIX 3

TC CALCULATIONS:
PROPOSED CONDITION

Chiquito Borrow Site
Proposed Condition
TC Data File

Project	Subarea	Area	%imp	Frequency	Soil Type	Length	Slope	Isohyet
CQT	1A	29.90	0.05	50	20	1,150	0.0895	5.8
CQT	2A	6.40	0.05	50	20	630	0.2700	5.8
CQT	3B	4.40	0.05	50	20	540	0.2800	5.8
CQT	4B	19.60	0.10	50	20	1,404	0.0560	5.8
CQT	5B	6.94	0.05	50	20	765	0.2100	5.8
CQT	6B	5.90	0.05	50	20	580	0.0600	5.8
CQT	7B	13.30	0.30	50	20	860	0.0600	5.8
CQT	9C	7.70	0.05	50	20	1,050	0.2800	5.8
CQT	10C	8.80	0.05	50	20	970	0.0600	5.8
CQT	11C	11.80	0.80	50	20	1,890	0.0080	5.8
CQT	12C	2.20	0.05	50	20	1,310	0.0240	5.8
CQT	13C	7.50	0.80	50	20	2,500	0.0070	5.8

Landmark Village
Proposed Condition
Tc Calculation Results

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft	Isohyet (in.)	Tc-calculated (min.)	Intensity (in./hr)	Cu	Cd	Flowrate (c	Tc Equation
CQT	1A	29.9	0.05	50	20	1150	0.0895	5.8	11	2.39	0.55	0.57	40.73	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	2A	6.4	0.05	50	20	630	0.27	5.8	6	3.18	0.62	0.63	12.82	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	3B	4.4	0.05	50	20	540	0.28	5.8	5	3.46	0.63	0.64	9.74	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	4B	19.6	0.1	50	20	1404	0.056	5.8	14	2.13	0.53	0.57	23.8	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	5B	6.94	0.05	50	20	765	0.21	5.8	7	2.95	0.6	0.62	12.69	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	6B	5.9	0.05	50	20	580	0.06	5.8	7	2.95	0.6	0.62	10.79	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	7B	13.3	0.3	50	20	860	0.06	5.8	9	2.63	0.57	0.67	23.44	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	9C	7.7	0.05	50	20	1050	0.28	5.8	8	2.77	0.59	0.61	13.01	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	10C	8.8	0.05	50	20	970	0.06	5.8	11	2.39	0.55	0.57	11.99	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	11C	11.8	0.8	50	20	1890	0.008	5.8	18	1.9	0.5	0.82	18.38	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	12C	2.2	0.05	50	20	1310	0.024	5.8	16	2	0.52	0.54	2.38	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
CQT	13C	7.5	0.8	50	20	2500	0.007	5.8	22	1.72	0.48	0.82	10.58	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Landmark Village - Newhall Ranch
Chiquito Canyon
Proposed Drainage Condition

Subarea	Area (Ac)	Calcs Tc (min)	I_{50} (in/hr)	Cu	Cd	Q_{50} (cfs)	Area Undeveloped (Ac)	K (1)	Q_{50b} (cfs) (2)	Q_{50bb} (cfs) (3)	$Q_{50b}+Q_d$ (cfs)	$Q_{50bb}+Q_d$ (cfs)	DP (cy) (4)	DP BASIN (cy)	$Q_{50design}$ (cfs)
1A	29.9	11	2.39	0.55	0.57	41	17.9	0.62	31.46	39.95	48	56	484		56
2A	6.4	6	3.18	0.62	0.63	13	2.9	0.60	7.22	9.17	14	16	79	563	16
3B	4.4	5	3.46	0.63	0.64	10	4.4	0.60	11.95	15.18	12	15	119		12
4B	19.6	14	2.13	0.53	0.57	24	11.9	0.63	18.52	23.52	28	33	321	440	28
5B	6.9	7	2.95	0.6	0.62	13	4.6	0.61	10.51	13.35	15	18	125	125	15
6B	5.9	7	2.95	0.6	0.62	11	0.0	0.61	0.00	0.00	11	11	0		11
7B	13.3	9	2.63	0.57	0.67	23	0.0	0.61	0.00	0.00	23	23	0		23
9C	7.7	8	2.77	0.59	0.61	13	1.9	0.61	4.09	5.20	14	15	52	52	14
10C	8.8	11	2.39	0.55	0.57	12	0.0	0.62	0.00	0.00	12	12	0		12
11C	11.8	18	1.9	0.5	0.82	18	0.0	0.63	0.00	0.00	18	18	0		18
12C	2.2	16	2	0.52	0.54	2	0.0	0.63	0.00	0.00	2	2	0		2
13C	7.5	22	1.72	0.48	0.82	11	0.0	0.64	0.00	0.00	11	11	0		11
Total	**124.4**					**190**			**84**	**106**	**208**	**230**	**1181**	**1181**	**219**

(1) - K = Burn Factor from Appendix G-2, K = 0.677*(I ^-0.102)
(2) - Qb = A*I*(1-K)+K*Qu
(3) - Qbb = 1.27*Qb
(4) - DP = A*27 cy (Zone 9)

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ADOBE Borrow Site
Proposed Condition
TC Data File

Project	Subarea	Area	%imp	Frequency	Soil Type	Length	Slope	Isohyet
ADB	1A	25.2	0	50	97	1280	0.150	5.8
ADB	2A	46.8	0	50	97	1130	0.110	5.8
ADB	3A	25.5	0	50	97	1080	0.050	5.8
ADB	4A	23.5	0	50	97	1140	0.052	5.8
ADB	5B	14.24	0	50	97	915	0.087	5.8
ADB	6B	40.7	0	50	97	2680	0.030	5.8
ADB	7C	17	0	50	97	1440	0.150	5.8
ADB	8C	6.6	0	50	97	415	0.150	5.8
ADB	9C	15.1	0	50	97	710	0.150	5.8

ADOBE CANYON
Proposed Condition
TC Calculation Results

Project	Subarea	Area (acres)	%imp	Frequency	Soil Type	Length (ft)	Slope (ft/ft	Isohyet (in.)	Tc-calculated (min.)	Intensity (in./hr)	Cu	Cd	Flowrate (c	Tc Equation
ADB	1A	25.2	0	50	97	1280	0.15	5.8	9	2.63	0.71	0.71	47.06	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	2A	46.8	0	50	97	1130	0.11	5.8	9	2.63	0.71	0.71	87.39	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	3A	25.5	0	50	97	1080	0.05	5.8	10	2.5	0.71	0.71	45.26	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	4A	23.5	0	50	97	1140	0.052	5.8	11	2.39	0.7	0.7	39.32	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	5B	14.24	0	50	97	915	0.087	5.8	8	2.77	0.72	0.72	28.4	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	6B	40.7	0	50	97	2680	0.03	5.8	21	1.76	0.64	0.64	45.84	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	7C	17	0	50	97	1440	0.15	5.8	10	2.5	0.71	0.71	30.18	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	8C	6.6	0	50	97	415	0.15	5.8	5	3.46	0.75	0.75	17.13	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135
ADB	9C	15.1	0	50	97	710	0.15	5.8	6	3.18	0.74	0.74	35.53	Tc=(10)^-0.507*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135

Landmark Village - Newhall Ranch
Adobe Canyon
Proposed Drainage Condition

Subarea	Area (Ac)	Calcs Tc (min)	I_{50} (in/hr)	Cu	Cd	Q_{50} (cfs)	Area Undeveloped (Ac)	K (1)	Q_{50b} (cfs) (2)	Q_{50bb} (cfs) (3)	$Q_{50b}+Q_d$ (cfs)	$Q_{50bb}+Q_d$ (cfs)	DP (cy) (4)	DP BASIN (cy)	$Q_{50design}$ (cfs)
ADB-1A	25.2	9	2.63	0.71	0.71	47	25.2	0.61	54	74	54	74	1361		54
ADB-2A	46.8	9	2.63	0.71	0.71	87	14.2	0.61	31	42	92	103	767		92
ADB-3A	25.5	10	2.50	0.71	0.71	45	12.78	0.62	26	36	49	58	690		49
ADB-4A	23.5	11	2.39	0.70	0.70	39	0.0	0.62	0	0	39	39	0	2818	39
ADB-5B	14.24	8	2.77	0.72	0.72	28	14.2	0.61	33	44	33	44	769		44
ADB-6B	40.7	21	1.76	0.64	0.64	46	32.0	0.64	43	59	53	69	1730		69
ADB-7C	17	10	2.50	0.71	0.71	30	17.0	0.62	35	47	35	47	918		47
ADB-8C	6.6	5	3.46	0.75	0.75	17	6.6	0.60	19	26	19	26	356		26
ADB-9C	15.1	6	3.18	0.74	0.74	36	8.3	0.60	22	30	38	46	447		46
Total	**214.6**					**376.11**			**264**	**359**	**412**	**506**	**7038**	**2818**	**466**

(1) - K = Burn Factor from Appendix G-2, K = 0.677*(I ^-0.102)
(2) - Qb = A*I*(1-K)+K*Qu
(3) - Qbb = 1.36*Qb
(4) - DP = A*54 cy (Zone 8)

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APPENDIX 4

LACDPW MORA CALCULATIONS: EXISTING CONDITION

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4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Existing
5 50300   17A  HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site
6 50300    1A 020  0   6 8A292  890 26500                      G1
6 50300    2A 020  0   4 6A292  200 01000
6 50300    3A  20  0   2 5A294  275 01000
6 50300    4A  20  0  1210A294  385 02000
6 50300    5B  20  0   4 5A292  530 32100
6 50300    6B  20  0  2515A294  835 02200
6 50300    7AB 20  0      A29
6 50300    8A  20  0   2 5A294  320 02000
6 50300    9A  20  0   3 5A294  900 01000
6 50300   10C  20  0  3214A292 1750 01500
6 50300   11C  20  0  1611A292 2800 01000
6 50300   12AC020         A29
6 50300   13D  20 65  1017A292 2370 01800
6 50300   14D  20 65  1210A292 1250 03200
6 50300   15AD 20         A29
6 50300   16A  20  0  1223A29
6 50300   17A 020  0    99A29                            1  2
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10-14-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Existing

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM RAIN ZONE	DAY 4 PCT IMPV
50300	1A	6.	10.	6.	10.	2	890.	0.26500	0.00	0.00	0.	20	8	A29	0.00
50300	2A	4.	8.	10.	17.	2	200.	0.01000	0.00	0.00	0.	20	6	A29	0.00
50300	3A	2.	4.	12.	18.	4	275.	0.01000	2.00	0.00	0.	20	5	A29	0.00
50300	4A	12.	17.	24.	34.	4	385.	0.02000	2.00	0.00	0.	20	10	A29	0.00
50300	5B	4.	9.	4.	9.	2	530.	0.32100	0.00	0.00	0.	20	5	A29	0.00
50300	6B	25.	27.	29.	35.	4	835.	0.02200	2.00	0.00	0.	20	15	A29	0.00
50300	7AB	29.	34.	53.	68.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	8A	2.	4.	55.	69.	4	320.	0.02000	2.75	0.00	0.	20	5	A29	0.00
50300	9A	3.	7.	58.	72.	4	900.	0.01000	3.00	0.00	0.	20	5	A29	0.00
50300	10C	32.	36.	32.	36.	2	1750.	0.01500	0.00	0.00	0.	20	14	A29	0.00
50300	11C	16.	21.	48.	40.	2	2800.	0.01000	0.00	0.00	0.	20	11	A29	0.00
50300	12AC	48.	29.	106.	78.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	13D	10.	16.	10.	16.	2	2370.	0.01800	0.00	0.00	0.	20	17	A29	0.65
50300	14D	12.	25.	22.	32.	2	1250.	0.03200	0.00	0.00	0.	20	10	A29	0.65
50300	15AD	22.	30.	128.	107.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	16A	12.	10.	140.	117.	0	0.	0.00000	0.00	0.00	0.	20	23	A29	0.00
50300	17A	0.	0.	140.	117.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site

HYDROGRAPH AT 50300 17A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	4.	200	4.	300	4.	400	5.
500	5.	600	5.	700	6.	800	6.	900	7.
1000	8.	1050	10.	1100	13.	1110	15.	1120	18.
1130	23.	1131	23.	1132	24.	1133	25.	1134	25.
1135	26.	1136	27.	1137	27.	1138	28.	1139	29.
1140	30.	1141	32.	1142	33.	1143	34.	1144	36.
1145	37.	1146	39.	1147	41.	1148	44.	1149	47.
1150	51.	1151	56.	1152	63.	1153	72.	1154	82.
1155	93.	1156	102.	1157	109.	1158	114.	1159	116.
1160	117.	1161	116.	1162	113.	1163	107.	1164	101.
1165	94.	1166	89.	1167	84.	1168	80.	1169	76.
1170	73.	1171	70.	1172	67.	1173	63.	1174	61.
1175	58.	1176	57.	1177	55.	1178	54.	1179	53.
1180	52.	1181	51.	1182	50.	1183	48.	1184	47.
1185	46.	1186	44.	1187	43.	1188	41.	1189	40.
1190	38.	1191	37.	1192	35.	1193	34.	1194	33.
1195	32.	1196	30.	1197	29.	1198	28.	1199	27.
1200	26.	1201	25.	1202	24.	1203	23.	1204	23.
1205	22.	1206	21.	1207	21.	1208	20.	1209	19.
1210	19.	1211	18.	1212	18.	1213	17.	1214	17.
1215	16.	1216	16.	1217	15.	1218	15.	1219	15.
1220	14.	1221	14.	1222	14.	1223	13.	1224	13.
1225	13.	1226	13.	1227	12.	1228	12.	1229	12.
1230	12.	1231	11.	1232	11.	1233	11.	1234	11.
1235	11.	1236	11.	1237	10.	1238	10.	1239	10.
1240	10.	1241	10.	1242	10.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	9.	1249	9.
1250	9.	1251	9.	1252	9.	1253	8.	1254	8.
1255	8.	1256	8.	1257	8.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	8.
1265	8.	1266	7.	1267	7.	1268	7.	1269	7.
1270	7.	1271	7.	1272	7.	1273	7.	1274	7.
1275	7.	1276	7.	1277	7.	1278	7.	1279	7.
1280	7.	1281	7.	1282	7.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	6.	1289	6.
1290	6.	1291	6.	1292	6.	1293	6.	1294	6.
1295	6.	1296	6.	1297	6.	1298	6.	1299	6.
1300	6.	1310	6.	1320	6.	1330	5.	1340	5.
1350	5.	1360	5.	1370	5.	1380	5.	1390	5.
1400	5.	1420	5.	1440	4.	1460	4.	1500	4.

Total Runoff = 17.108 Acre-Ft.
Peak Q = 117 CFS
Time to Peak Q = 1160 Minutes

```
4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Existing
5 50300   17A  HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site
6 50300    1A 198  0   6 8A292  890 26500                      G1
6 50300    2A 198  0   4 6A292  200 01000
6 50300    3A 198  0   2 5A294  275 01000
6 50300    4A 198  0  1210A294  385 02000
6 50300    5B 198  0   4 5A292  530 32100
6 50300    6B 198  0  2515A294  835 02200
6 50300    7AB198  0      A29
6 50300    8A 198  0   2 5A294  320 02000
6 50300    9A 198  0   3 5A294  900 01000
6 50300   10C 198  0  3214A292 1750 01500
6 50300   11C 198  0  1611A292 2800 01000
6 50300   12AC198         A29
6 50300   13D 198 65  1017A292 2370 01800
6 50300   14D 198 65  1210A292 1250 03200
6 50300   15AD198         A29
6 50300   16A 198  0  1223A29
6 50300   17A 198  0    99A29                            1  2
```

10-19-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Existing

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
50300	1A	6.	13.	6.	13.	2	890.	0.26500	0.00	0.00	0.	198	8	A29	0.00
50300	2A	4.	10.	10.	21.	2	200.	0.01000	0.00	0.00	0.	198	6	A29	0.00
50300	3A	2.	5.	12.	24.	4	275.	0.01000	2.00	0.00	0.	198	5	A29	0.00
50300	4A	12.	22.	24.	45.	4	385.	0.02000	2.25	0.00	0.	198	10	A29	0.00
50300	5B	4.	11.	4.	11.	2	530.	0.32100	0.00	0.00	0.	198	5	A29	0.00
50300	6B	25.	37.	29.	47.	4	835.	0.02200	2.25	0.00	0.	198	15	A29	0.00
50300	7AB	29.	46.	53.	90.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	8A	2.	5.	55.	93.	4	320.	0.02000	3.00	0.00	0.	198	5	A29	0.00
50300	9A	3.	8.	58.	96.	4	900.	0.01000	3.50	0.00	0.	198	5	A29	0.00
50300	10C	32.	49.	32.	49.	2	1750.	0.01500	0.00	0.00	0.	198	14	A29	0.00
50300	11C	16.	28.	48.	59.	2	2800.	0.01000	0.00	0.00	0.	198	11	A29	0.00
50300	12AC	48.	46.	106.	115.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	13D	10.	17.	10.	17.	2	2370.	0.01800	0.00	0.00	0.	198	17	A29	0.65
50300	14D	12.	27.	22.	34.	2	1250.	0.03200	0.00	0.00	0.	198	10	A29	0.65
50300	15AD	22.	33.	128.	147.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	16A	12.	14.	140.	160.	0	0.	0.00000	0.00	0.00	0.	198	23	A29	0.00
50300	17A	0.	0.	140.	160.	0	0.	0.00000	0.00	0.00	0.	198	99	A29	0.00

HYDROGRAPH AT OUTLET BASIN 600 and Chiquito site

HYDROGRAPH AT 50300 17A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	10.	200	11.	300	11.	400	12.
500	12.	600	13.	700	15.	800	16.	900	18.
1000	22.	1050	26.	1100	33.	1110	36.	1120	41.
1130	47.	1131	47.	1132	48.	1133	49.	1134	49.
1135	50.	1136	51.	1137	52.	1138	53.	1139	54.
1140	55.	1141	57.	1142	58.	1143	60.	1144	62.
1145	64.	1146	67.	1147	69.	1148	72.	1149	76.
1150	81.	1151	88.	1152	97.	1153	109.	1154	121.
1155	134.	1156	145.	1157	153.	1158	158.	1159	160.
1160	160.	1161	158.	1162	153.	1163	146.	1164	139.
1165	132.	1166	126.	1167	121.	1168	117.	1169	112.
1170	108.	1171	104.	1172	99.	1173	96.	1174	92.
1175	90.	1176	88.	1177	86.	1178	85.	1179	83.
1180	81.	1181	79.	1182	77.	1183	75.	1184	72.
1185	70.	1186	68.	1187	65.	1188	63.	1189	61.
1190	59.	1191	57.	1192	55.	1193	53.	1194	52.
1195	50.	1196	48.	1197	47.	1198	45.	1199	44.
1200	43.	1201	42.	1202	41.	1203	40.	1204	39.
1205	38.	1206	37.	1207	36.	1208	35.	1209	34.
1210	34.	1211	33.	1212	32.	1213	32.	1214	31.
1215	31.	1216	30.	1217	29.	1218	29.	1219	29.
1220	28.	1221	28.	1222	27.	1223	27.	1224	26.
1225	26.	1226	26.	1227	25.	1228	25.	1229	25.
1230	24.	1231	24.	1232	24.	1233	24.	1234	23.
1235	23.	1236	23.	1237	23.	1238	22.	1239	22.
1240	22.	1241	22.	1242	22.	1243	21.	1244	21.
1245	21.	1246	21.	1247	21.	1248	20.	1249	20.
1250	20.	1251	20.	1252	20.	1253	20.	1254	20.
1255	19.	1256	19.	1257	19.	1258	19.	1259	19.
1260	19.	1261	19.	1262	18.	1263	18.	1264	18.
1265	18.	1266	18.	1267	18.	1268	18.	1269	18.
1270	18.	1271	17.	1272	17.	1273	17.	1274	17.
1275	17.	1276	17.	1277	17.	1278	17.	1279	17.
1280	17.	1281	17.	1282	17.	1283	16.	1284	16.
1285	16.	1286	16.	1287	16.	1288	16.	1289	16.
1290	16.	1291	16.	1292	16.	1293	16.	1294	16.
1295	16.	1296	15.	1297	15.	1298	15.	1299	15.
1300	15.	1310	15.	1320	14.	1330	14.	1340	13.
1350	13.	1360	13.	1370	12.	1380	12.	1390	12.
1400	12.	1420	11.	1440	11.	1460	10.	1500	10.

Total Runoff = 37.567 Acre-Ft.
Peak Q = 160 CFS
Time to Peak Q = 1159 Minutes

```
4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - ADOBE Existing
5 50300   11A  HYDROGRAPH AT OUTLET BASIN AREAs A, B, And c
6 50300    1A 097  035.809A292 1755 61400                      G1
6 50300    2A 097  0  4010A292 1035 05900
6 50300    3A  97  0  2411A292  890 34000
6 50300    4B  97  016.711A292 1560 13800
6 50300    5B  97  039.916A292 2490 09200
6 50300    6AB097          A29
6 50300    7C 097  027.412A292  720 01300
6 50300    8C 097  012.710A292 1250 01200
6 50300    9C 097  016.6 8A292 1150 01000
6 50300   10AC097          A29
6 50300   11A 097  0    99A29                            1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - ADOBE Existing

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
50300	1A	36.	67.	36.	67.	2	1755.	0.61400	0.00	0.00	0.	97	9	A29	0.00
50300	2A	40.	71.	76.	136.	2	1035.	0.05900	0.00	0.00	0.	97	10	A29	0.00
50300	3A	24.	40.	100.	171.	2	890.	0.34000	0.00	0.00	0.	97	11	A29	0.00
50300	4B	17.	28.	17.	28.	2	1560.	0.13800	0.00	0.00	0.	97	11	A29	0.00
50300	5B	40.	53.	57.	80.	2	2490.	0.09200	0.00	0.00	0.	97	16	A29	0.00
50300	6AB	57.	77.	157.	240.	0	0.	0.00000	0.00	0.00	0.	97	0	A29	0.00
50300	7C	27.	43.	27.	43.	2	720.	0.01300	0.00	0.00	0.	97	12	A29	0.00
50300	8C	13.	23.	40.	61.	2	1250.	0.01200	0.00	0.00	0.	97	10	A29	0.00
50300	9C	17.	34.	57.	60.	2	1150.	0.01000	0.00	0.00	0.	97	8	A29	0.00
50300	10AC	57.	58.	214.	287.	0	0.	0.00000	0.00	0.00	0.	97	0	A29	0.00
50300	11A	0.	0.	214.	287.	0	0.	0.00000	0.00	0.00	0.	97	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY
HYDROGRAPH AT OUTLET BASIN AREAs A, B, And c

HYDROGRAPH AT 50300 11A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	3.	400	3.
500	4.	600	4.	700	4.	800	4.	900	5.
1000	6.	1050	10.	1100	21.	1110	28.	1120	40.
1130	54.	1131	55.	1132	57.	1133	59.	1134	60.
1135	62.	1136	64.	1137	66.	1138	68.	1139	71.
1140	73.	1141	76.	1142	79.	1143	83.	1144	87.
1145	91.	1146	96.	1147	101.	1148	107.	1149	115.
1150	126.	1151	140.	1152	160.	1153	184.	1154	212.
1155	238.	1156	260.	1157	275.	1158	284.	1159	287.
1160	284.	1161	273.	1162	257.	1163	238.	1164	218.
1165	199.	1166	183.	1167	170.	1168	157.	1169	146.
1170	135.	1171	125.	1172	114.	1173	105.	1174	97.
1175	90.	1176	83.	1177	77.	1178	72.	1179	66.
1180	62.	1181	57.	1182	53.	1183	49.	1184	46.
1185	43.	1186	40.	1187	37.	1188	35.	1189	33.
1190	31.	1191	29.	1192	27.	1193	26.	1194	24.
1195	23.	1196	22.	1197	21.	1198	19.	1199	18.
1200	17.	1201	17.	1202	16.	1203	15.	1204	14.
1205	14.	1206	13.	1207	13.	1208	12.	1209	12.
1210	11.	1211	11.	1212	10.	1213	10.	1214	10.
1215	10.	1216	9.	1217	9.	1218	9.	1219	9.
1220	8.	1221	8.	1222	8.	1223	8.	1224	8.
1225	8.	1226	7.	1227	7.	1228	7.	1229	7.
1230	7.	1231	7.	1232	7.	1233	7.	1234	6.
1235	6.	1236	6.	1237	6.	1238	6.	1239	6.
1240	6.	1241	6.	1242	6.	1243	6.	1244	6.
1245	6.	1246	6.	1247	6.	1248	6.	1249	5.
1250	5.	1251	5.	1252	5.	1253	5.	1254	5.
1255	5.	1256	5.	1257	5.	1258	5.	1259	5.
1260	5.	1261	5.	1262	5.	1263	5.	1264	5.
1265	5.	1266	5.	1267	5.	1268	5.	1269	5.
1270	5.	1271	5.	1272	5.	1273	5.	1274	5.
1275	5.	1276	5.	1277	5.	1278	5.	1279	5.
1280	5.	1281	5.	1282	4.	1283	4.	1284	4.
1285	4.	1286	4.	1287	4.	1288	4.	1289	4.
1290	4.	1291	4.	1292	4.	1293	4.	1294	4.
1295	4.	1296	4.	1297	4.	1298	4.	1299	4.
1300	4.	1310	4.	1320	4.	1330	4.	1340	4.
1350	4.	1360	4.	1370	3.	1380	3.	1390	3.
1400	3.	1420	3.	1440	3.	1460	3.	1500	3.

Total Runoff = 20.132 Acre-Ft.
Peak Q = 287 CFS
Time to Peak Q = 1159 Minutes

APPENDIX 5

LACDPW MORA CALCULATIONS
PROPOSED CONDITION

```
4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Proposed
5 50300   15A  HYDROGRAPH AT OUTLET BASIN AREAs A
6 50300    1A 020  529.911A292 1220 01600                     G1
6 50300    2A 020  5 6.4 6A292  880 02700
6 50300    3B  20  5 4.4 5A292 1410 05300
6 50300    4B 020  519.614A292  760 00200
6 50300    5B 020  5 6.9 7A294  132 04000
6 50300    6B 020  5 5.9 7A29
6 50300    7B  20 3013.3 9A294  917 00200
6 50300    8AB020         A29
6 50300    9C 020  5 7.7 8A294  540 00400
6 50300   10C  20  5 8.811A294  250 00100
6 50300   11C  20 8011.818A294 1610 00100
6 50300   12C  20  5 2.216A294  360 00100
6 50300   13C  20 80 7.522A294  630 00100
6 50300   14AC020         A29
6 50300   15A 020  0    50A29                             1  2
```

9-11-2008 SITE LICENSEE: PSOMAS AND ASSOCIATES PAGE 1

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Proposed STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
50300	1A	30.	42.	30.	42.	2	1220.	0.01600	0.00	0.00	0.	20	11	A29	0.05
50300	2A	6.	12.	36.	38.	2	880.	0.02700	0.00	0.00	0.	20	6	A29	0.05
50300	3B	4.	9.	4.	9.	2	1410.	0.05300	0.00	0.00	0.	20	5	A29	0.05
50300	4B	20.	24.	24.	29.	2	760.	0.00200	0.00	0.00	0.	20	14	A29	0.05
50300	5B	7.	13.	31.	23.	4	132.	0.04000	2.00	0.00	0.	20	7	A29	0.05
50300	6B	6.	11.	37.	30.	0	0.	0.00000	0.00	0.00	0.	20	7	A29	0.05
50300	7B	13.	24.	50.	54.	4	917.	0.00200	3.75	0.00	0.	20	9	A29	0.30
50300	8AB	50.	50.	86.	82.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	9C	8.	13.	8.	13.	4	540.	0.00400	2.00	0.00	0.	20	8	A29	0.05
50300	10C	9.	12.	17.	25.	4	250.	0.00100	3.25	0.00	0.	20	11	A29	0.05
50300	11C	12.	20.	29.	45.	4	1610.	0.00100	4.00	0.00	0.	20	18	A29	0.80
50300	12C	2.	2.	31.	39.	4	360.	0.00100	3.75	0.00	0.	20	16	A29	0.05
50300	13C	7.	11.	38.	47.	4	630.	0.00100	4.00	0.00	0.	20	22	A29	0.80
50300	14AC	38.	46.	124.	109.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	15A	0.	0.	124.	109.	0	0.	0.00000	0.00	0.00	0.	20	50	A29	0.00

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 50300 15A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	5.	200	5.	300	5.	400	6.
500	6.	600	6.	700	7.	800	7.	900	8.
1000	10.	1050	12.	1100	16.	1110	17.	1120	20.
1130	25.	1131	26.	1132	26.	1133	27.	1134	27.
1135	28.	1136	29.	1137	30.	1138	30.	1139	31.
1140	32.	1141	33.	1142	35.	1143	36.	1144	37.
1145	39.	1146	40.	1147	42.	1148	44.	1149	46.
1150	49.	1151	52.	1152	57.	1153	64.	1154	73.
1155	82.	1156	91.	1157	98.	1158	104.	1159	108.
1160	109.	1161	109.	1162	108.	1163	106.	1164	105.
1165	104.	1166	103.	1167	102.	1168	101.	1169	100.
1170	99.	1171	97.	1172	95.	1173	93.	1174	90.
1175	87.	1176	83.	1177	79.	1178	74.	1179	69.
1180	65.	1181	60.	1182	56.	1183	52.	1184	49.
1185	45.	1186	42.	1187	40.	1188	37.	1189	35.
1190	33.	1191	31.	1192	29.	1193	28.	1194	27.
1195	25.	1196	24.	1197	23.	1198	22.	1199	21.
1200	21.	1201	20.	1202	19.	1203	19.	1204	18.
1205	17.	1206	17.	1207	16.	1208	16.	1209	16.
1210	15.	1211	15.	1212	14.	1213	14.	1214	14.
1215	14.	1216	13.	1217	13.	1218	13.	1219	13.
1220	12.	1221	12.	1222	12.	1223	12.	1224	12.
1225	11.	1226	11.	1227	11.	1228	11.	1229	11.
1230	11.	1231	11.	1232	10.	1233	10.	1234	10.
1235	10.	1236	10.	1237	10.	1238	10.	1239	10.
1240	10.	1241	10.	1242	10.	1243	9.	1244	9.
1245	9.	1246	9.	1247	9.	1248	9.	1249	9.
1250	9.	1251	9.	1252	9.	1253	9.	1254	9.
1255	9.	1256	9.	1257	9.	1258	8.	1259	8.
1260	8.	1261	8.	1262	8.	1263	8.	1264	8.
1265	8.	1266	8.	1267	8.	1268	8.	1269	8.
1270	8.	1271	8.	1272	8.	1273	8.	1274	8.
1275	8.	1276	8.	1277	8.	1278	8.	1279	8.
1280	8.	1281	8.	1282	8.	1283	7.	1284	7.
1285	7.	1286	7.	1287	7.	1288	7.	1289	7.
1290	7.	1291	7.	1292	7.	1293	7.	1294	7.
1295	7.	1296	7.	1297	7.	1298	7.	1299	7.
1300	7.	1310	7.	1320	7.	1330	6.	1340	6.
1350	6.	1360	6.	1370	6.	1380	6.	1390	6.
1400	5.	1420	5.	1440	5.	1460	5.	1500	5.

Total Runoff = 19.355 Acre-Ft.
Peak Q = 109 CFS
Time to Peak Q = 1160 Minutes

```
 4LADEPTH.RDT
 5 50300    1A  50 YR River Village - Newhall Ranch - Chiquito Proposed BURNED
 5 50300   15A  HYDROGRAPH AT OUTLET BASIN AREAs A
 6 50300    1A 198  538.011A292 1220 01600                    G1
 6 50300    2A 198  5 8.0 6A292  880 02700
 6 50300    3B 198  5 4.4 5A292 1410 05300
 6 50300    4B 198  519.614A292  760 00200
 6 50300    5B 198  5 6.9 7A294  132 04000
 6 50300    6B 198  5 5.9 7A29
 6 50300    7B 198 3013.3 9A294  917 00200
 6 50300    8AB198         A29
 6 50300    9C 198  5 7.7 8A294  540 00400
 6 50300   10C 198  5 8.811A294  250 00100
 6 50300   11C  20 8011.818A294 1610 00100
 6 50300   12C  20  5 2.216A294  360 00100
 6 50300   13C  20 80 7.522A294  630 00100
 6 50300   14AC020         A29
 6 50300   15A 020  0    50A29                             1  2

**NOTE**
        THE AREA FOR SUBAREAS 1A AND 2A HAVE BEEN INCREASED TO ACCOUNT FOR BURNED AND BULKED FLOWRATE.
        AREA 1A ACTUAL AREA IS 30.0 ACRES
        AREA 2A ACTUAL AREA IS 6.0 ACRES
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11-11-2009 SITE LICENSEE: PSOMAS AND ASSOCIATES

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - Chiquito Proposed BURNED

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	STORM DAY 4 RAIN ZONE	PCT IMPV
50300	1A	38.	68.	38.	68.	2	1220.	0.01600	0.00	0.00	0.	198	11	A29	0.05
50300	2A	8.	20.	46.	67.	2	880.	0.02700	0.00	0.00	0.	198	6	A29	0.05
50300	3B	4.	11.	4.	11.	2	1410.	0.05300	0.00	0.00	0.	198	5	A29	0.05
50300	4B	20.	31.	24.	38.	2	760.	0.00200	0.00	0.00	0.	198	14	A29	0.05
50300	5B	7.	16.	31.	34.	4	132.	0.04000	2.00	0.00	0.	198	7	A29	0.05
50300	6B	6.	14.	37.	42.	0	0.	0.00000	0.00	0.00	0.	198	7	A29	0.05
50300	7B	13.	28.	50.	70.	4	917.	0.00200	4.25	0.00	0.	198	9	A29	0.30
50300	8AB	50.	65.	96.	127.	0	0.	0.00000	0.00	0.00	0.	198	0	A29	0.00
50300	9C	8.	17.	8.	17.	4	540.	0.00400	2.25	0.00	0.	198	8	A29	0.05
50300	10C	9.	16.	17.	32.	4	250.	0.00100	3.50	0.00	0.	198	11	A29	0.05
50300	11C	12.	20.	29.	52.	4	1610.	0.00100	4.25	0.00	0.	20	18	A29	0.80
50300	12C	2.	2.	31.	46.	4	360.	0.00100	4.00	0.00	0.	20	16	A29	0.05
50300	13C	7.	11.	38.	54.	4	630.	0.00100	4.25	0.00	0.	20	22	A29	0.80
50300	14AC	38.	52.	134.	159.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	15A	0.	0.	134.	159.	0	0.	0.00000	0.00	0.00	0.	20	50	A29	0.00

NOTE

THE AREA FOR SUBAREAS 1A AND 2A HAVE BEEN INCREASED TO ACCOUNT FOR BURNED AND BULKED FLOWRATE.
AREA 1A ACTUAL AREA IS 30.0 ACRES
AREA 2A ACTUAL AREA IS 6.0 ACRES

HYDROGRAPH AT OUTLET BASIN AREAs A

HYDROGRAPH AT 50300 15A STORM DAY 4 REDUCTION FACTOR = 1.000

TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	10.	200	11.	300	11.	400	12.
500	12.	600	13.	700	14.	800	16.	900	18.
1000	22.	1050	26.	1100	33.	1110	36.	1120	41.
1130	47.	1131	48.	1132	48.	1133	49.	1134	50.
1135	51.	1136	52.	1137	53.	1138	54.	1139	55.
1140	56.	1141	57.	1142	59.	1143	60.	1144	62.
1145	64.	1146	66.	1147	68.	1148	71.	1149	74.
1150	77.	1151	82.	1152	90.	1153	100.	1154	112.
1155	124.	1156	136.	1157	146.	1158	153.	1159	157.
1160	159.	1161	158.	1162	156.	1163	153.	1164	150.
1165	148.	1166	146.	1167	144.	1168	141.	1169	138.
1170	134.	1171	131.	1172	127.	1173	123.	1174	118.
1175	113.	1176	107.	1177	101.	1178	95.	1179	89.
1180	84.	1181	79.	1182	74.	1183	70.	1184	66.
1185	62.	1186	59.	1187	56.	1188	53.	1189	51.
1190	49.	1191	47.	1192	45.	1193	43.	1194	42.
1195	40.	1196	39.	1197	38.	1198	37.	1199	36.
1200	35.	1201	34.	1202	33.	1203	33.	1204	32.
1205	31.	1206	31.	1207	30.	1208	29.	1209	29.
1210	28.	1211	28.	1212	28.	1213	27.	1214	27.
1215	26.	1216	26.	1217	26.	1218	25.	1219	25.
1220	25.	1221	24.	1222	24.	1223	24.	1224	24.
1225	23.	1226	23.	1227	23.	1228	23.	1229	22.
1230	22.	1231	22.	1232	22.	1233	22.	1234	21.
1235	21.	1236	21.	1237	21.	1238	21.	1239	21.
1240	20.	1241	20.	1242	20.	1243	20.	1244	20.
1245	20.	1246	20.	1247	19.	1248	19.	1249	19.
1250	19.	1251	19.	1252	19.	1253	19.	1254	19.
1255	18.	1256	18.	1257	18.	1258	18.	1259	18.
1260	18.	1261	18.	1262	18.	1263	18.	1264	18.
1265	17.	1266	17.	1267	17.	1268	17.	1269	17.
1270	17.	1271	17.	1272	17.	1273	17.	1274	17.
1275	17.	1276	17.	1277	16.	1278	16.	1279	16.
1280	16.	1281	16.	1282	16.	1283	16.	1284	16.
1285	16.	1286	16.	1287	16.	1288	16.	1289	16.
1290	15.	1291	15.	1292	15.	1293	15.	1294	15.
1295	15.	1296	15.	1297	15.	1298	15.	1299	15.
1300	15.	1310	14.	1320	14.	1330	13.	1340	13.
1350	13.	1360	12.	1370	12.	1380	12.	1390	12.
1400	11.	1420	11.	1440	11.	1460	10.	1500	10.

Total Runoff = 37.255 Acre-Ft.
Peak Q = 159 CFS
Time to Peak Q = 1160 Minutes

```
4LADEPTH.RDT
5 50300    1A  50 YR River Village - Newhall Ranch - ADOBE Proposed
5 50300   12A  HYDROGRAPH AT OUTLET BASIN AREAs A, B, and C
6 50300    1A 020  025.2 9A292 1390 05750                      G1
6 50300    2A 020  046.8 9A292  160 05000
6 50300    3A  20  025.510A292  550 05000
6 50300    4A  20  023.511A292 1000 03000
6 50300    5B  20  014.2 8A292 2680 03000
6 50300    6B  20  040.721A29
6 50300    7AB020          A29
6 50300    8C 020  017.010A292  740 01500
6 50300    9C 020  0 6.605A292 1300 01500
6 50300   10C 020  015.1 6A292 1100 01500
6 50300   11AC020          A29
6 50300   12A 020  0    99A29                             1  2
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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT PROG FO601A

MODIFIED RATIONAL METHOD HYDROLOGY

50 YR River Village - Newhall Ranch - ADOBE Proposed STORM DAY 4

LOCATION		SUBAREA AREA	SUBAREA Q	TOTAL AREA	TOTAL Q	CONV TYPE	CONV LNGTH	CONV SLOPE	CONV SIZE	CONV Z	CONTROL Q	SOIL NAME	TC	RAIN ZONE	PCT IMPV
50300	1A	25.	38.	25.	38.	2	1390.	0.05750	0.00	0.00	0.	20	9	A29	0.00
50300	2A	47.	71.	72.	99.	2	160.	0.05000	0.00	0.00	0.	20	9	A29	0.00
50300	3A	25.	36.	97.	133.	2	550.	0.05000	0.00	0.00	0.	20	10	A29	0.00
50300	4A	23.	31.	120.	161.	2	1000.	0.03000	0.00	0.00	0.	20	11	A29	0.00
50300	5B	14.	23.	14.	23.	2	2680.	0.03000	0.00	0.00	0.	20	8	A29	0.00
50300	6B	41.	36.	55.	47.	0	0.	0.00000	0.00	0.00	0.	20	21	A29	0.00
50300	7AB	55.	47.	175.	202.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	8C	17.	24.	17.	24.	2	740.	0.01500	0.00	0.00	0.	20	10	A29	0.00
50300	9C	7.	15.	24.	26.	2	1300.	0.01500	0.00	0.00	0.	20	5	A29	0.00
50300	10C	15.	29.	39.	38.	2	1100.	0.01500	0.00	0.00	0.	20	6	A29	0.00
50300	11AC	39.	30.	214.	232.	0	0.	0.00000	0.00	0.00	0.	20	0	A29	0.00
50300	12A	0.	0.	214.	232.	0	0.	0.00000	0.00	0.00	0.	20	99	A29	0.00

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
MODIFIED RATIONAL METHOD HYDROLOGY
HYDROGRAPH AT OUTLET BASIN AREAs A, B, and c

HYDROGRAPH AT 50300 12A STORM DAY 4 REDUCTION FACTOR = 1.000

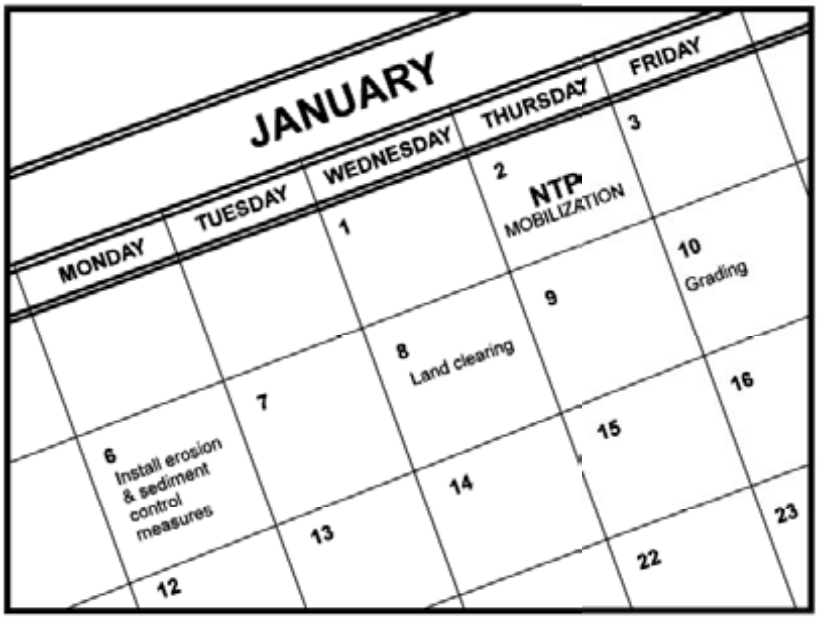
TIME	Q	TIME	Q	TIME	Q	TIME	Q	TIME	Q
0	0.	100	3.	200	3.	300	3.	400	4.
500	4.	600	4.	700	4.	800	5.	900	5.
1000	6.	1050	8.	1100	13.	1110	15.	1120	21.
1130	30.	1131	31.	1132	32.	1133	33.	1134	34.
1135	36.	1136	37.	1137	38.	1138	40.	1139	42.
1140	43.	1141	46.	1142	48.	1143	51.	1144	54.
1145	57.	1146	61.	1147	65.	1148	70.	1149	78.
1150	87.	1151	98.	1152	112.	1153	132.	1154	153.
1155	176.	1156	199.	1157	216.	1158	227.	1159	232.
1160	231.	1161	225.	1162	212.	1163	196.	1164	178.
1165	161.	1166	144.	1167	129.	1168	117.	1169	107.
1170	95.	1171	85.	1172	75.	1173	67.	1174	59.
1175	54.	1176	49.	1177	45.	1178	42.	1179	39.
1180	36.	1181	34.	1182	32.	1183	30.	1184	28.
1185	26.	1186	25.	1187	23.	1188	22.	1189	21.
1190	20.	1191	19.	1192	18.	1193	17.	1194	16.
1195	16.	1196	15.	1197	14.	1198	14.	1199	13.
1200	13.	1201	12.	1202	12.	1203	11.	1204	11.
1205	11.	1206	10.	1207	10.	1208	10.	1209	9.
1210	9.	1211	9.	1212	9.	1213	8.	1214	8.
1215	8.	1216	8.	1217	8.	1218	8.	1219	7.
1220	7.	1221	7.	1222	7.	1223	7.	1224	7.
1225	7.	1226	7.	1227	7.	1228	7.	1229	6.
1230	6.	1231	6.	1232	6.	1233	6.	1234	6.
1235	6.	1236	6.	1237	6.	1238	6.	1239	6.
1240	6.	1241	6.	1242	6.	1243	6.	1244	6.
1245	5.	1246	5.	1247	5.	1248	5.	1249	5.
1250	5.	1251	5.	1252	5.	1253	5.	1254	5.
1255	5.	1256	5.	1257	5.	1258	5.	1259	5.
1260	5.	1261	5.	1262	5.	1263	5.	1264	5.
1265	5.	1266	5.	1267	5.	1268	5.	1269	5.
1270	5.	1271	5.	1272	5.	1273	5.	1274	5.
1275	5.	1276	5.	1277	5.	1278	5.	1279	5.
1280	5.	1281	4.	1282	4.	1283	4.	1284	4.
1285	4.	1286	4.	1287	4.	1288	4.	1289	4.
1290	4.	1291	4.	1292	4.	1293	4.	1294	4.
1295	4.	1296	4.	1297	4.	1298	4.	1299	4.
1300	4.	1310	4.	1320	4.	1330	4.	1340	4.
1350	4.	1360	4.	1370	4.	1380	4.	1390	3.
1400	3.	1420	3.	1440	3.	1460	3.	1500	3.

Total Runoff = 16.302 Acre-Ft.
Peak Q = 232 CFS
Time to Peak Q = 1159 Minutes

APPENDIX 6

EROSION AND SEDIMENT CONTROL BMP FACT SHEETS

Scheduling EC-1



Objectives

EC	Erosion Control	☑
SE	Sediment Control	☒
TR	Tracking Control	☒
WE	Wind Erosion Control	☒
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

Scheduling is the development of a written plan that includes sequencing of construction activities and the implementation of BMPs such as erosion control and sediment control while taking local climate (rainfall, wind, etc.) into consideration. The purpose is to reduce the amount and duration of soil exposed to erosion by wind, rain, runoff, and vehicle tracking, and to perform the construction activities and control practices in accordance with the planned schedule.

Suitable Applications

Proper sequencing of construction activities to reduce erosion potential should be incorporated into the schedule of every construction project especially during rainy season. Use of other, more costly yet less effective, erosion and sediment control BMPs may often be reduced through proper construction sequencing.

Limitations

- Environmental constraints such as nesting season prohibitions reduce the full capabilities of this BMP.

Implementation

- Avoid rainy periods. Schedule major grading operations during dry months when practical. Allow enough time before rainfall begins to stabilize the soil with vegetation or physical means or to install sediment trapping devices.
- Plan the project and develop a schedule showing each phase of construction. Clearly show how the rainy season relates to soil

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disturbing and re-stabilization activities. Incorporate the construction schedule into the SWPPP.

- Include on the schedule, details on the rainy season implementation and deployment of:
  - Erosion control BMPs
  - Sediment control BMPs
  - Tracking control BMPs
  - Wind erosion control BMPs
  - Non-stormwater BMPs
  - Waste management and materials pollution control BMPs
- Include dates for activities that may require non-stormwater discharges such as dewatering, sawcutting, grinding, drilling, boring, crushing, blasting, painting, hydro-demolition, mortar mixing, pavement cleaning, etc.
- Work out the sequencing and timetable for the start and completion of each item such as site clearing and grubbing, grading, excavation, paving, foundation pouring utilities installation, etc., to minimize the active construction area during the rainy season.
  - Sequence trenching activities so that most open portions are closed before new trenching begins.
  - Incorporate staged seeding and re-vegetation of graded slopes as work progresses.
  - Schedule establishment of permanent vegetation during appropriate planting time for specified vegetation.
- Non-active areas should be stabilized as soon as practical after the cessation of soil disturbing activities or one day prior to the onset of precipitation.
- Monitor the weather forecast for rainfall.
- When rainfall is predicted, adjust the construction schedule to allow the implementation of soil stabilization and sediment treatment controls on all disturbed areas prior to the onset of rain.
- Be prepared year round to deploy erosion control and sediment control BMPs. Erosion may be caused during dry seasons by un-seasonal rainfall, wind, and vehicle tracking. Keep the site stabilized year round, and retain and maintain rainy season sediment trapping devices in operational condition.
- Apply permanent erosion control to areas deemed substantially complete during the project's defined seeding window.

Costs

Construction scheduling to reduce erosion may increase other construction costs due to reduced economies of scale in performing site grading. The cost effectiveness of scheduling techniques should be compared with the other less effective erosion and sedimentation controls to achieve a cost effective balance.

Inspection and Maintenance

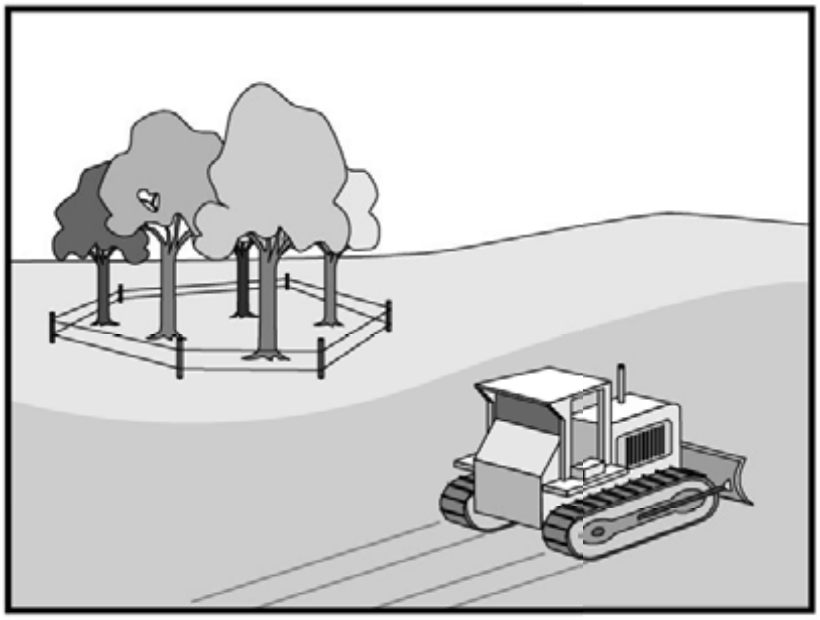
- Verify that work is progressing in accordance with the schedule. If progress deviates, take corrective actions.
- Amend the schedule when changes are warranted.
- Amend the schedule prior to the rainy season to show updated information on the deployment and implementation of construction site BMPs.

References

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stormwater Management for Construction Activities Developing Pollution Prevention Plans and Best Management Practices (EPA 832-R-92-005), U.S. Environmental Protection Agency, Office of Water, September 1992.

Preservation Of Existing Vegetation EC-2



Objectives

EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

Carefully planned preservation of existing vegetation minimizes the potential of removing or injuring existing trees, vines, shrubs, and grasses that protect soil from erosion.

Suitable Applications

Preservation of existing vegetation is suitable for use on most projects. Large project sites often provide the greatest opportunity for use of this BMP. Suitable applications include the following:

- Areas within the site where no construction activity occurs, or occurs at a later date. This BMP is especially suitable to multi year projects where grading can be phased.
- Areas where natural vegetation exists and is designated for preservation. Such areas often include steep slopes, watercourse, and building sites in wooded areas.
- Areas where local, state, and federal government require preservation, such as vernal pools, wetlands, marshes, certain oak trees, etc. These areas are usually designated on the plans, or in the specifications, permits, or environmental documents.
- Where vegetation designated for ultimate removal can be temporarily preserved and be utilized for erosion control and sediment control.

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EC-2 Preservation Of Existing Vegetation

Limitations

- Requires forward planning by the owner/developer, contractor, and design staff.
- Limited opportunities for use when project plans do not incorporate existing vegetation into the site design.
- For sites with diverse topography, it is often difficult and expensive to save existing trees while grading the site satisfactory for the planned development.

Implementation

The best way to prevent erosion is to not disturb the land. In order to reduce the impacts of new development and redevelopment, projects may be designed to avoid disturbing land in sensitive areas of the site (e.g., natural watercourses, steep slopes), and to incorporate unique or desirable existing vegetation into the site's landscaping plan. Clearly marking and leaving a buffer area around these unique areas during construction will help to preserve these areas as well as take advantage of natural erosion prevention and sediment trapping.

Existing vegetation to be preserved on the site must be protected from mechanical and other injury while the land is being developed. The purpose of protecting existing vegetation is to ensure the survival of desirable vegetation for shade, beautification, and erosion control. Mature vegetation has extensive root systems that help to hold soil in place, thus reducing erosion. In addition, vegetation helps keep soil from drying rapidly and becoming susceptible to erosion. To effectively save existing vegetation, no disturbances of any kind should be allowed within a defined area around the vegetation. For trees, no construction activity should occur within the drip line of the tree.

Timing

- Provide for preservation of existing vegetation prior to the commencement of clearing and grubbing operations or other soil disturbing activities in areas where no construction activity is planned or will occur at a later date.

Design and Layout

- Mark areas to be preserved with temporary fencing. Include sufficient setback to protect roots.
  - Orange colored plastic mesh fencing works well.
  - Use appropriate fence posts and adequate post spacing and depth to completely support the fence in an upright position.
- Locate temporary roadways, stockpiles, and layout areas to avoid stands of trees, shrubs, and grass.
- Consider the impact of grade changes to existing vegetation and the root zone.
- Maintain existing irrigation systems where feasible. Temporary irrigation may be required.
- Instruct employees and subcontractors to honor protective devices. Prohibit heavy equipment, vehicular traffic, or storage of construction materials within the protected area.

Costs

There is little cost associated with preserving existing vegetation if properly planned during the project design, and these costs may be offset by aesthetic benefits that enhance property values. During construction, the cost for preserving existing vegetation will likely be less than the cost of applying erosion and sediment controls to the disturbed area. Replacing vegetation inadvertently destroyed during construction can be extremely expensive, sometimes in excess of $10,000 per tree.

Inspection and Maintenance

During construction, the limits of disturbance should remain clearly marked at all times. Irrigation or maintenance of existing vegetation should be described in the landscaping plan. If damage to protected trees still occurs, maintenance guidelines described below should be followed:

- Verify that protective measures remain in place. Restore damaged protection measures immediately.
- Serious tree injuries shall be attended to by an arborist.
- Damage to the crown, trunk, or root system of a retained tree shall be repaired immediately.
- Trench as far from tree trunks as possible, usually outside of the tree drip line or canopy. Curve trenches around trees to avoid large roots or root concentrations. If roots are encountered, consider tunneling under them. When trenching or tunneling near or under trees to be retained, place tunnels at least 18 in. below the ground surface, and not below the tree center to minimize impact on the roots.
- Do not leave tree roots exposed to air. Cover exposed roots with soil as soon as possible. If soil covering is not practical, protect exposed roots with wet burlap or peat moss until the tunnel or trench is ready for backfill.
- Cleanly remove the ends of damaged roots with a smooth cut.
- Fill trenches and tunnels as soon as possible. Careful filling and tamping will eliminate air spaces in the soil, which can damage roots.
- If bark damage occurs, cut back all loosened bark into the undamaged area, with the cut tapered at the top and bottom and drainage provided at the base of the wood. Limit cutting the undamaged area as much as possible.
- Aerate soil that has been compacted over a trees root zone by punching holes 12 in. deep with an iron bar, and moving the bar back and forth until the soil is loosened. Place holes 18 in. apart throughout the area of compacted soil under the tree crown.
- Fertilization
  - Fertilize stressed or damaged broadleaf trees to aid recovery.
  - Fertilize trees in the late fall or early spring.

- Apply fertilizer to the soil over the feeder roots and in accordance with label instructions, but never closer than 3 ft to the trunk. Increase the fertilized area by one-fourth of the crown area for conifers that have extended root systems.

- Retain protective measures until all other construction activity is complete to avoid damage during site cleanup and stabilization.

References

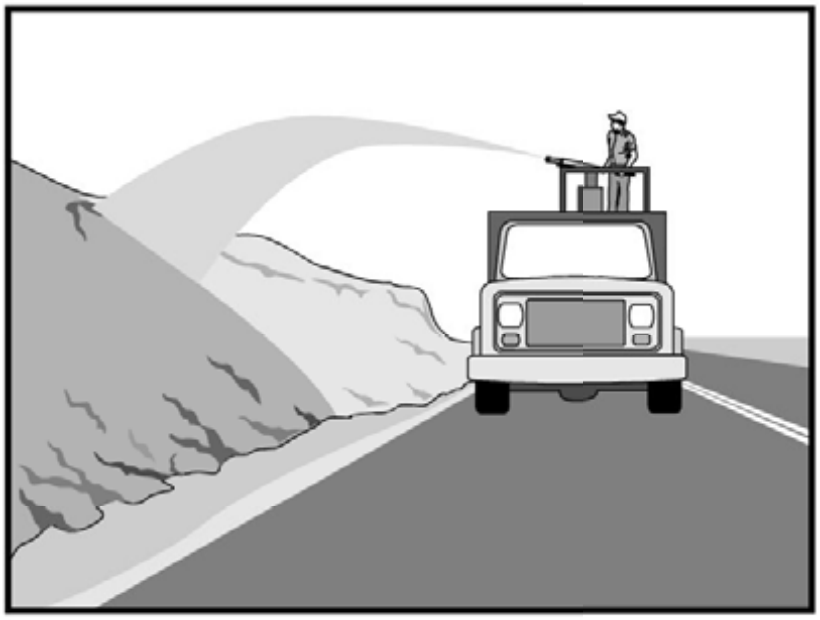
County of Sacramento Tree Preservation Ordinance, September 1981.

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stormwater Management of the Puget Sound Basin, Technical Manual, Publication #91-75, Washington State Department of Ecology, February 1992.

Water Quality Management Plan for The Lake Tahoe Region, Volume II, Handbook of Management Practices, Tahoe Regional Planning Agency, November 1988.

Hydraulic Mulch EC-3



Objectives		
EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	☒
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-4 Hydroseeding
EC-5 Soil Binders
EC-6 Straw Mulch
EC-7 Geotextiles and Mats
EC-8 Wood Mulching

Description and Purpose

Hydraulic mulch consists of applying a mixture of shredded wood fiber or a hydraulic matrix, and a stabilizing emulsion or tackifier with hydro-mulching equipment, which temporarily protects exposed soil from erosion by raindrop impact or wind.

Suitable Applications

Hydraulic mulch is suitable for soil disturbed areas requiring temporary protection until permanent stabilization is established, and disturbed areas that will be re-disturbed following an extended period of inactivity.

Limitations

Wood fiber hydraulic mulches are generally short lived and need 24 hours to dry before rainfall occurs to be effective. May require a second application in order to remain effective for an entire rainy season.

Implementation

- Prior to application, roughen embankment and fill areas by rolling with a crimping or punching type roller or by track walking. Track walking shall only be used where other methods are impractical.
- To be effective, hydraulic matrices require 24 hours to dry before rainfall occurs.
- Avoid mulch over spray onto roads, sidewalks, drainage channels, existing vegetation, etc.

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- Paper based hydraulic mulches alone shall not be used for erosion control.

Hydraulic Mulches

Wood fiber mulch can be applied alone or as a component of hydraulic matrices. Wood fiber applied alone is typically applied at the rate of 2,000 to 4,000 lb/acre. Wood fiber mulch is manufactured from wood or wood waste from lumber mills or from urban sources.

Hydraulic Matrices

Hydraulic matrices include a mixture of wood fiber and acrylic polymer or other tackifier as binder. Apply as a liquid slurry using a hydraulic application machine (i.e., hydro seeder) at the following minimum rates, or as specified by the manufacturer to achieve complete coverage of the target area: 2,000 to 4,000 lb/acre wood fiber mulch, and 5 to 10% (by weight) of tackifier (acrylic copolymer, guar, psyllium, etc.)

Bonded Fiber Matrix

Bonded fiber matrix (BFM) is a hydraulically applied system of fibers and adhesives that upon drying forms an erosion resistant blanket that promotes vegetation, and prevents soil erosion. BFMs are typically applied at rates from 3,000 lb/acre to 4,000 lb/acre based on the manufacturer's recommendation. A biodegradable BFM is composed of materials that are 100% biodegradable. The binder in the BFM should also be biodegradable and should not dissolve or disperse upon re-wetting. Typically, biodegradable BFMs should not be applied immediately before, during or immediately after rainfall if the soil is saturated. Depending on the product, BFMs typically require 12 to 24 hours to dry and become effective.

Costs

Average cost for installation of wood fiber mulch is $900/acre. Average cost for installation of BFM is $5,500/acre.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Areas where erosion is evident shall be repaired and BMPs re-applied as soon as possible. Care should be exercised to minimize the damage to protected areas while making repairs, as any area damaged will require re-application of BMPs.
- Maintain an unbroken, temporary mulched ground cover throughout the period of construction when the soils are not being reworked.

References

Controlling Erosion of Construction Sites Agricultural Information #347, U.S. Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) (formerly Soil Conservation Service – SCS).

Guides for Erosion and Sediment Control in California, USDA Soils Conservation Service, January 1991.

Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

Sedimentation and Erosion Control, An Inventory of Current Practices Draft, US EPA, April 1990.

Soil Erosion by Water, Agriculture Information Bulletin #513, U.S. Department of Agriculture, Soil Conservation Service.

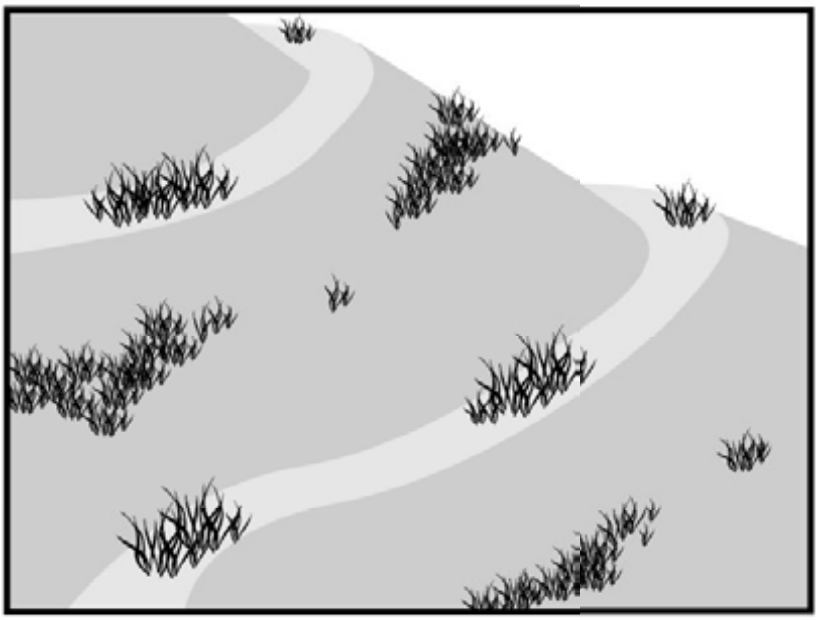
Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Guidance Document: Soil Stabilization for Temporary Slopes, State of California Department of Transportation (Caltrans), November 1999

Stormwater Management of the Puget Sound Basin, Technical Manual, Publication #91-75, Washington State Department of Ecology, February 1992.

Water Quality Management Plan for the Lake Tahoe Region, Volume II, Handbook of Management Practices, Tahoe Regional Planning Agency, November 1988.

Hydroseeding EC-4



Objectives		
EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	☒
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-3 Hydraulic Mulch
EC-5 Soil Binders
EC-6 Straw Mulch
EC-7 Geotextiles and Mats
EC-8 Wood Mulching

Description and Purpose

Hydroseeding typically consists of applying a mixture of wood fiber, seed, fertilizer, and stabilizing emulsion with hydro-mulch equipment, to temporarily protect exposed soils from erosion by water and wind.

Suitable Applications

Hydroseeding is suitable for soil disturbed areas requiring temporary protection until permanent stabilization is established, and disturbed areas that will be re-disturbed following an extended period of inactivity.

Limitations

- Hydroseeding may be used alone only when there is sufficient time in the season to ensure adequate vegetation establishment and coverage to provide adequate erosion control. Otherwise, hydroseeding must be used in conjunction with mulching (i.e., straw mulch).
- Steep slopes are difficult to protect with temporary seeding.
- Temporary seeding may not be appropriate in dry periods without supplemental irrigation.
- Temporary vegetation may have to be removed before permanent vegetation is applied.
- Temporary vegetation is not appropriate for short term inactivity.

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Implementation

In order to select appropriate hydroseeding mixtures, an evaluation of site conditions shall be performed with respect to:

- Soil conditions
- Site topography
- Season and climate
- Vegetation types
- Maintenance requirements
- Sensitive adjacent areas
- Water availability
- Plans for permanent vegetation

The local office of the U.S.D.A. Natural Resources Conservation Service (NRCS) is an excellent source of information on appropriate seed mixes.

The following steps shall be followed for implementation:

- Avoid use of hydroseeding in areas where the BMP would be incompatible with future earthwork activities and would have to be removed.
- Hydroseeding can be accomplished using a multiple step or one step process. The multiple step process ensures maximum direct contact of the seeds to soil. When the one step process is used to apply the mixture of fiber, seed, etc., the seed rate shall be increased to compensate for all seeds not having direct contact with the soil.
- Prior to application, roughen the area to be seeded with the furrows trending along the contours.
- Apply a straw mulch to keep seeds in place and to moderate soil moisture and temperature until the seeds germinate and grow.
- All seeds shall be in conformance with the California State Seed Law of the Department of Agriculture. Each seed bag shall be delivered to the site sealed and clearly marked as to species, purity, percent germination, dealer's guarantee, and dates of test. The container shall be labeled to clearly reflect the amount of Pure Live Seed (PLS) contained. All legume seed shall be pellet inoculated. Inoculant sources shall be species specific and shall be applied at a rate of 2 lb of inoculant per 100 lb seed.
- Commercial fertilizer shall conform to the requirements of the California Food and Agricultural Code. Fertilizer shall be pelleted or granular form.
- Follow up applications shall be made as needed to cover weak spots and to maintain adequate soil protection.
- Avoid over spray onto roads, sidewalks, drainage channels, existing vegetation, etc.

Costs

Average cost for installation and maintenance may vary from as low as $300 per acre for flat slopes and stable soils, to $1600 per acre for moderate to steep slopes and/or erosive soils.

Hydroseeding		Installed Cost per Acre
High Density	Ornamentals	$400 - $1600
	Turf Species	$350
	Bunch Grasses	$300 - $1300
Fast Growing	Annual	$350 - $650
	Perennial	$300 - $800
Non-Competing	Native	$300 - $1600
	Non-Native	$400 - $500
Sterile	Cereal Grain	$500

Source: Caltrans Guidance for Soil Stabilization for Temporary Slopes, Nov. 1999

Inspection and Maintenance

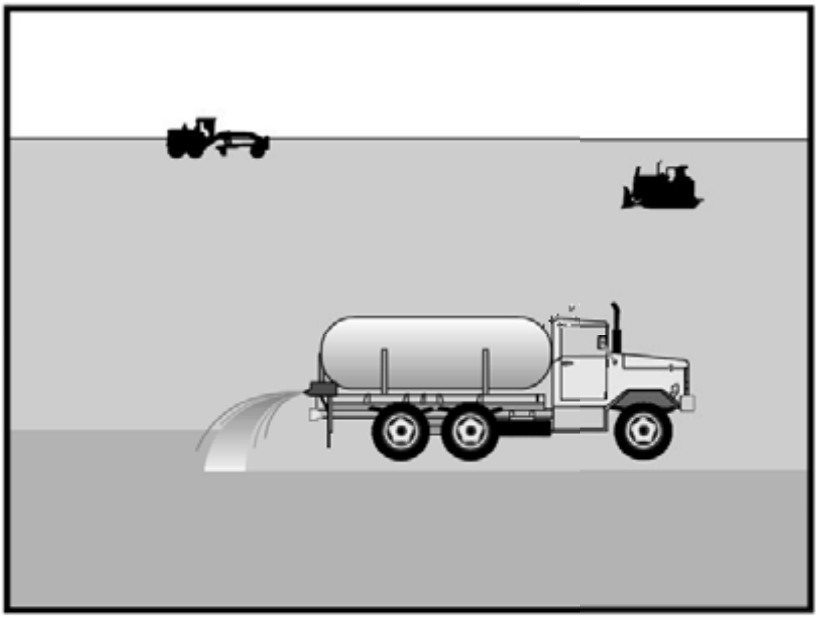
- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Areas where erosion is evident shall be repaired and BMPs re-applied as soon as possible. Care should be exercised to minimize the damage to protected areas while making repairs, as any area damaged will require re-application of BMPs.
- Where seeds fail to germinate, or they germinate and die, the area must be re-seeded, fertilized, and mulched within the planting season, using not less than half the original application rates.
- Irrigation systems, if applicable, should be inspected daily while in use to identify system malfunctions and line breaks. When line breaks are detected, the system must be shut down immediately and breaks repaired before the system is put back into operation.
- Irrigation systems shall be inspected for complete coverage and adjusted as needed to maintain complete coverage.

References

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Guidance Document: Soil Stabilization for Temporary Slopes, State of California Department of Transportation (Caltrans), November 1999.

Soil Binders EC-5



Objectives		
EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	☒
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-3 Hydraulic Mulch
EC-4 Hydroseeding
EC-6 Straw Mulch
EC-7 Geotextiles and Mats
EC-8 Wood Mulching

Description and Purpose

Soil binders consist of applying and maintaining a soil stabilizer to exposed soil surfaces. Soil binders are materials applied to the soil surface to temporarily prevent water induced erosion of exposed soils on construction sites. Soil binders also prevent wind erosion.

Suitable Applications

Soil binders are typically applied to disturbed areas requiring short term temporary protection. Because soil binders can often be incorporated into the work, they are a good alternative to mulches in areas where grading activities will soon resume. Soil binders are also suitable for use on stockpiles.

Limitations

- Soil binders are temporary in nature and may need reapplication.
- Soil binders require a minimum curing time until fully effective, as prescribed by the manufacturer. Curing time may be 24 hours or longer. Soil binders may need reapplication after a storm event.
- Soil binders will generally experience spot failures during heavy rainfall events. If runoff penetrates the soil at the top of a slope treated with a soil binder, it is likely that the runoff will undercut the stabilized soil layer and discharge at a point further down slope.

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- Soil binders do not hold up to pedestrian or vehicular traffic across treated areas.
- Soil binders may not penetrate soil surfaces made up primarily of silt and clay, particularly when compacted.
- Some soil binders may not perform well with low relative humidity. Under rainy conditions, some agents may become slippery or leach out of the soil.
- Soil binders may not cure if low temperatures occur within 24 hours of application.
- The water quality impacts of soil binders are relatively unknown and some may have water quality impacts due to their chemical makeup.
- A sampling and analysis plan must be incorporated into the SWPPP as soil binders could be a source of non-visible pollutants.

Implementation

General Considerations

- Regional soil types will dictate appropriate soil binders to be used.
- A soil binder must be environmentally benign (non-toxic to plant and animal life), easy to apply, easy to maintain, economical, and should not stain paved or painted surfaces. Soil binders should not pollute stormwater.
- Some soil binders may not be compatible with existing vegetation.
- Performance of soil binders depends on temperature, humidity, and traffic across treated areas.
- Avoid over spray onto roads, sidewalks, drainage channels, existing vegetation, etc.

Selecting a Soil Binder

Properties of common soil binders used for erosion control are provided on Table 1 at the end of this BMP. Use Table 1 to select an appropriate soil binder. Refer to WE-1, Wind Erosion Control, for dust control soil binders.

Factors to consider when selecting a soil binder include the following:

- Suitability to situation - Consider where the soil binder will be applied, if it needs a high resistance to leaching or abrasion, and whether it needs to be compatible with any existing vegetation. Determine the length of time soil stabilization will be needed, and if the soil binder will be placed in an area where it will degrade rapidly. In general, slope steepness is not a discriminating factor for the listed soil binders.
- Soil types and surface materials - Fines and moisture content are key properties of surface materials. Consider a soil binder's ability to penetrate, likelihood of leaching, and ability to form a surface crust on the surface materials.
- Frequency of application - The frequency of application can be affected by subgrade conditions, surface type, climate, and maintenance schedule. Frequent applications could

lead to high costs. Application frequency may be minimized if the soil binder has good penetration, low evaporation, and good longevity. Consider also that frequent application will require frequent equipment clean up.

Plant-Material Based (Short Lived) Binders

Guar: Guar is a non-toxic, biodegradable, natural galactomannan based hydrocolloid treated with dispersant agents for easy field mixing. It should be mixed with water at the rate of 11 to 15 lb per 1,000 gallons. Recommended minimum application rates are as follows:

Application Rates for Guar Soil Stabilizer

Slope (H:V):	Flat	4:1	3:1	2:1	1:1
lb/acre:	40	45	50	60	70

Psyllium: Psyllium is composed of the finely ground muciloid coating of plantago seeds that is applied as a dry powder or in a wet slurry to the surface of the soil. It dries to form a firm but rewettable membrane that binds soil particles together but permits germination and growth of seed. Psyllium requires 12 to 18 hours drying time. Application rates should be from 80 to 200 lb/acre, with enough water in solution to allow for a uniform slurry flow.

Starch: Starch is non-ionic, cold water soluble (pre-gelatinized) granular cornstarch. The material is mixed with water and applied at the rate of 150 lb/acre. Approximate drying time is 9 to 12 hours.

Plant-Material Based (Long Lived) Binders

Pitch and Rosin Emulsion: Generally, a non-ionic pitch and rosin emulsion has a minimum solids content of 48%. The rosin should be a minimum of 26% of the total solids content. The soil stabilizer should be non-corrosive, water dilutable emulsion that upon application cures to a water insoluble binding and cementing agent. For soil erosion control applications, the emulsion is diluted and should be applied as follows:

- For clayey soil: 5 parts water to 1 part emulsion
- For sandy soil: 10 parts water to 1 part emulsion

Application can be by water truck or hydraulic seeder with the emulsion and product mixture applied at the rate specified by the manufacturer.

Polymeric Emulsion Blend Binders

Acrylic Copolymers and Polymers: Polymeric soil stabilizers should consist of a liquid or solid polymer or copolymer with an acrylic base that contains a minimum of 55% solids. The polymeric compound should be handled and mixed in a manner that will not cause foaming or should contain an anti-foaming agent. The polymeric emulsion should not exceed its shelf life or expiration date; manufacturers should provide the expiration date. Polymeric soil stabilizer should be readily miscible in water, non-injurious to seed or animal life, non-flammable, should provide surface soil stabilization for various soil types without totally inhibiting water infiltration, and should not re-emulsify when cured. The applied compound should air cure within a maximum of 36 to 48 hours. Liquid copolymer should be diluted at a rate of 10 parts water to 1 part polymer and the mixture applied to soil at a rate of 1,175 gallons/acre.

Liquid Polymers of Methacrylates and Acrylates: This material consists of a tackifier/sealer that is a liquid polymer of methacrylates and acrylates. It is an aqueous 100% acrylic emulsion blend of 40% solids by volume that is free from styrene, acetate, vinyl, ethoxylated surfactants or silicates. For soil stabilization applications, it is diluted with water in accordance with manufacturer's recommendations, and applied with a hydraulic seeder at the rate of 20 gallons/acre. Drying time is 12 to 18 hours after application.

Copolymers of Sodium Acrylates and Acrylamides: These materials are non-toxic, dry powders that are copolymers of sodium acrylate and acrylamide. They are mixed with water and applied to the soil surface for erosion control at rates that are determined by slope gradient:

Slope Gradient (H:V)	lb/acre
Flat to 5:1	3.0 – 5.0
5:1 to 3:1	5.0 – 10.0
2:2 to 1:1	10.0 – 20.0

Poly-Acrylamide and Copolymer of Acrylamide: Linear copolymer polyacrylamide is packaged as a dry flowable solid. When used as a stand alone stabilizer, it is diluted at a rate of 11lb/1,000 gal of water and applied at the rate of 5.0 lb/acre.

Hydro-Colloid Polymers: Hydro-Colloid Polymers are various combinations of dry flowable poly-acrylamides, copolymers and hydro-colloid polymers that are mixed with water and applied to the soil surface at rates of 55 to 60 lb/acre. Drying times are 0 to 4 hours.

Cementitious-Based Binders

Gypsum: This is a formulated gypsum based product that readily mixes with water and mulch to form a thin protective crust on the soil surface. It is composed of high purity gypsum that is ground, calcined and processed into calcium sulfate hemihydrate with a minimum purity of 86%. It is mixed in a hydraulic seeder and applied at rates 4,000 to 12,000 lb/acre. Drying time is 4 to 8 hours.

Applying Soil Binders

After selecting an appropriate soil binder, the untreated soil surface must be prepared before applying the soil binder. The untreated soil surface must contain sufficient moisture to assist the agent in achieving uniform distribution. In general, the following steps should be followed:

- Follow manufacturer's written recommendations for application rates, pre-wetting of application area, and cleaning of equipment after use.
- Prior to application, roughen embankment and fill areas.
- Consider the drying time for the selected soil binder and apply with sufficient time before anticipated rainfall. Soil binders should not be applied during or immediately before rainfall.
- Avoid over spray onto roads, sidewalks, drainage channels, sound walls, existing vegetation, etc.

- Soil binders should not be applied to frozen soil, areas with standing water, under freezing or rainy conditions, or when the temperature is below 40°F during the curing period.
- More than one treatment is often necessary, although the second treatment may be diluted or have a lower application rate.
- Generally, soil binders require a minimum curing time of 24 hours before they are fully effective. Refer to manufacturer's instructions for specific cure time.
- For liquid agents:
  - Crown or slope ground to avoid ponding.
  - Uniformly pre-wet ground at 0.03 to 0.3 gal/yd² or according to manufacturer's recommendations.
  - Apply solution under pressure. Overlap solution 6 to 12 in.
  - Allow treated area to cure for the time recommended by the manufacturer; typically at least 24 hours.
  - Apply second treatment before first treatment becomes ineffective, using 50% application rate.
  - In low humidities, reactivate chemicals by re-wetting with water at 0.1 to 0.2 gal/yd².

Costs

Costs vary according to the soil stabilizer selected for implementation. The following are approximate costs:

Soil Binder	Cost per Acre
Plant-Material Based (Short Lived) Binders	$400
Plant-Material Based (Long Lived) Binders	$1,200
Polymeric Emulsion Blend Binders	$400 (1)
Cementitious-Based Binders	$800

(1) $1,200 for Acrylic polymers and copolymers

Source: Caltrans Guidance for Soil Stabilization for Temporary Slopes, Nov. 1999

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Areas where erosion is evident shall be repaired and BMPs re-applied as soon as possible. Care should be exercised to minimize the damage to protected areas while making repairs, as any area damaged will require re-application of BMPs.
- Reapply the selected soil binder as needed to maintain effectiveness.

References

Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

Sedimentation and Erosion Control, An Inventory of Current Practices Draft, US EPA, April 1990.

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Guidance Document: Soil Stabilization for Temporary Slopes, State of California Department of Transportation (Caltrans), November 1999.

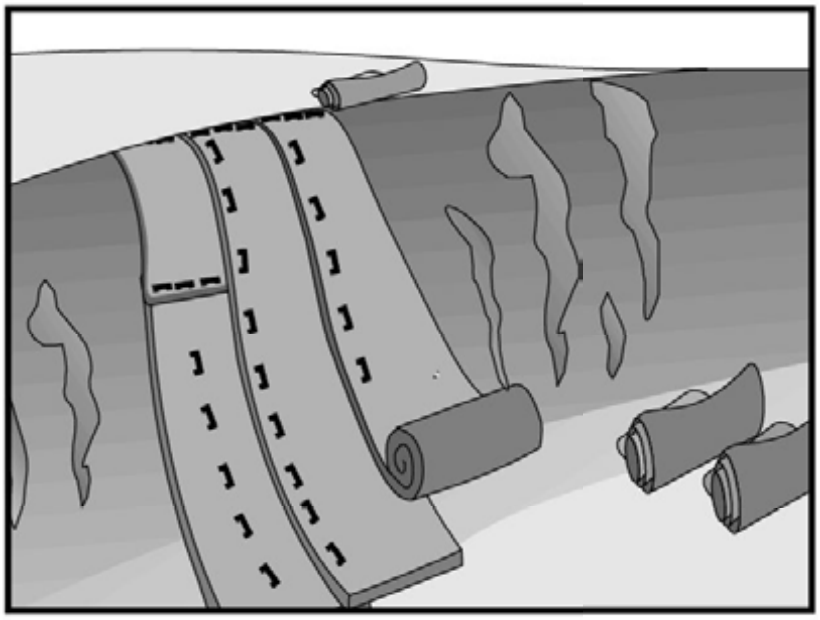
Stormwater Management for Construction Activities, Developing Pollution Prevention Plans and Best Management Practices, EPA 832-R-92005; USEPA, April 1992.

Table 1 Properties of Soil Binders for Erosion Control

Evaluation Criteria	Binder Type			
	Plant Material Based (Short Lived)	Plant Material Based (Long Lived)	Polymeric Emulsion Blends	Cementitious-Based Binders
Relative Cost	Low	Low	Low	Low
Resistance to Leaching	High	High	Low to Moderate	Moderate
Resistance to Abrasion	Moderate	Low	Moderate to High	Moderate to High
Longevity	Short to Medium	Medium	Medium to Long	Medium
Minimum Curing Time before Rain	9 to 18 hours	19 to 24 hours	0 to 24 hours	4 to 8 hours
Compatibility with Existing Vegetation	Good	Poor	Poor	Poor
Mode of Degradation	Biodegradable	Biodegradable	Photodegradable/ Chemically Degradable	Photodegradable/ Chemically Degradable
Labor Intensive	No	No	No	No
Specialized Application Equipment	Water Truck or Hydraulic Mulcher	Water Truck or Hydraulic Mulcher	Water Truck or Hydraulic Mulcher	Water Truck or Hydraulic Mulcher
Liquid/Powder	Powder	Liquid	Liquid/Powder	Powder
Surface Crusting	Yes, but dissolves on rewetting	Yes	Yes, but dissolves on rewetting	Yes
Clean Up	Water	Water	Water	Water
Erosion Control Application Rate	Varies (1)	Varies (1)	Varies (1)	4,000 to 12,000 lbs/acre

(1) See Implementation for specific rates.

Geotextiles and Mats EC-7



Objectives

EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	3
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-3 Hydraulic Mulch

EC-4 Hydroseeding

EC-5 Soil Binders

EC-6 Straw Mulch

EC-8 Wood Mulching

Description and Purpose

Mattings of natural materials are used to cover the soil surface to reduce erosion from rainfall impact, hold soil in place, and absorb and hold moisture near the soil surface. Additionally, matting may be used to stabilize soils until vegetation is established.

Suitable Applications

Mattings are commonly applied on short, steep slopes where erosion hazard is high and vegetation will be slow to establish. Mattings are also used on stream banks where moving water at velocities between 3 ft/s and 6 ft/s are likely to wash out new vegetation, and in areas where the soil surface is disturbed and where existing vegetation has been removed Matting may also be used when seeding cannot occur (e.g., late season construction and/or the arrival of an early rain season). Erosion control matting should be considered when the soils are fine grained and potentially erosive. These measures should be considered in the following situations.

- Steep slopes, generally steeper than 3:1 (H:V)
- Slopes where the erosion potential is high
- Slopes and disturbed soils where mulch must be anchored
- Disturbed areas where plants are slow to develop
- Channels with flows exceeding 3.3 ft/s

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- Channels to be vegetated
- Stockpiles
- Slopes adjacent to water bodies of Environmentally Sensitive Areas (ESAs)

Limitations

- Properly installed mattings provide excellent erosion control but do so at relatively high cost. This high cost typically limits the use of mattings to areas of concentrated channel flow and steep slopes.
- Mattings are more costly than other BMP practices, limiting their use to areas where other BMPs are ineffective (e.g. channels, steep slopes).
- Installation is critical and requires experienced contractors. The contractor should install the matting material in such a manner that continuous contact between the material and the soil occurs.
- Geotextiles and Mats may delay seed germination, due to reduction in soil temperature.
- Blankets and mats are generally not suitable for excessively rocky sites or areas where the final vegetation will be mowed (since staples and netting can catch in mowers).
- Blankets and mats must be removed and disposed of prior to application of permanent soil stabilization measures.
- Plastic sheeting is easily vandalized, easily torn, photodegradable, and must be disposed of at a landfill.
- Plastic results in 100% runoff, which may cause serious erosion problems in the areas receiving the increased flow.
- The use of plastic should be limited to covering stockpiles or very small graded areas for short periods of time (such as through one imminent storm event) until alternative measures, such as seeding and mulching, may be installed.
- Geotextiles, mats, plastic covers, and erosion control covers have maximum flow rate limitations; consult the manufacturer for proper selection.
- Not suitable for areas that have heavy foot traffic (tripping hazard) – e.g., pad areas around buildings under construction.

Implementation

Material Selection

Organic matting materials have been found to be effective where re-vegetation will be provided by re-seeding. The choice of matting should be based on the size of area, side slopes, surface conditions such as hardness, moisture, weed growth, and availability of materials.

The following natural and synthetic mattings are commonly used:

Geotextiles

- Material should be a woven polypropylene fabric with minimum thickness of 0.06 in., minimum width of 12 ft and should have minimum tensile strength of 150 lbs (warp), 80 lbs (fill) in conformance with the requirements in ASTM Designation: D 4632. The permittivity of the fabric should be approximately 0.07 sec^{-1} in conformance with the requirements in ASTM Designation: D4491. The fabric should have an ultraviolet (UV) stability of 70 percent in conformance with the requirements in ASTM designation: D4355. Geotextile blankets must be secured in place with wire staples or sandbags and by keying into tops of slopes to prevent infiltration of surface waters under geotextile. Staples should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.

- Geotextiles may be reused if they are suitable for the use intended.

Plastic Covers

- Plastic sheeting should have a minimum thickness of 6 mils, and must be keyed in at the top of slope and firmly held in place with sandbags or other weights placed no more than 10 ft apart. Seams are typically taped or weighted down their entire length, and there should be at least a 12 in. to 24 in. overlap of all seams. Edges should be embedded a minimum of 6 in. in soil.

- All sheeting must be inspected periodically after installation and after significant rainstorms to check for erosion, undermining, and anchorage failure. Any failures must be repaired immediately. If washout or breakages occur, the material should be re-installed after repairing the damage to the slope.

Erosion Control Blankets/Mats

- Biodegradable rolled erosion control products (RECPs) are typically composed of jute fibers, curled wood fibers, straw, coconut fiber, or a combination of these materials. In order for an RECP to be considered 100% biodegradable, the netting, sewing or adhesive system that holds the biodegradable mulch fibers together must also be biodegradable.

  - **Jute** is a natural fiber that is made into a yarn that is loosely woven into a biodegradable mesh. It is designed to be used in conjunction with vegetation and has longevity of approximately one year. The material is supplied in rolled strips, which should be secured to the soil with U-shaped staples or stakes in accordance with manufacturers' recommendations.

  - **Excelsior** (curled wood fiber) blanket material should consist of machine produced mats of curled wood excelsior with 80 percent of the fiber 6 in. or longer. The excelsior blanket should be of consistent thickness. The wood fiber must be evenly distributed over the entire area of the blanket. The top surface of the blanket should be covered with a photodegradable extruded plastic mesh. The blanket should be smolder resistant without the use of chemical additives and should be non-toxic and non-injurious to plant and animal life. Excelsior blankets should be furnished in rolled strips, a minimum of 48 in. wide, and should have an average weight of 0.8 lb/yd^2, ±10 percent, at the time of manufacture. Excelsior blankets must be secured in place with wire staples. Staples

should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.

- **Straw blanket** should be machine produced mats of straw with a lightweight biodegradable netting top layer. The straw should be attached to the netting with biodegradable thread or glue strips. The straw blanket should be of consistent thickness. The straw should be evenly distributed over the entire area of the blanket. Straw blanket should be furnished in rolled strips a minimum of 6.5 ft wide, a minimum of 80 ft long and a minimum of 0.5 lb/yd². Straw blankets must be secured in place with wire staples. Staples should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.

- **Wood fiber blanket** is composed of biodegradable fiber mulch with extruded plastic netting held together with adhesives. The material is designed to enhance re-vegetation. The material is furnished in rolled strips, which must be secured to the ground with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Coconut fiber blanket** should be a machine produced mat of 100 percent coconut fiber with biodegradable netting on the top and bottom. The coconut fiber should be attached to the netting with biodegradable thread or glue strips. The coconut fiber blanket should be of consistent thickness. The coconut fiber should be evenly distributed over the entire area of the blanket. Coconut fiber blanket should be furnished in rolled strips with a minimum of 6.5 ft wide, a minimum of 80 ft. long and a minimum of 0.5 lb/yd². Coconut fiber blankets must be secured in place with wire staples. Staples should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.

- **Coconut fiber mesh** is a thin permeable membrane made from coconut or corn fiber that is spun into a yarn and woven into a biodegradable mat. It is designed to be used in conjunction with vegetation and typically has longevity of several years. The material is supplied in rolled strips, which must be secured to the soil with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Straw coconut fiber blanket** should be machine produced mats of 70 percent straw and 30 percent coconut fiber with a biodegradable netting top layer and a biodegradable bottom net. The straw and coconut fiber should be attached to the netting with biodegradable thread or glue strips. The straw coconut fiber blanket should be of consistent thickness. The straw and coconut fiber should be evenly distributed over the entire area of the blanket. Straw coconut fiber blanket should be furnished in rolled strips a minimum of 6.5 ft wide, a minimum of 80 ft long and a minimum of 0.5 lb/yd². Straw coconut fiber blankets must be secured in place with wire staples. Staples should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.

- Non-biodegradable RECPs are typically composed of polypropylene, polyethylene, nylon or other synthetic fibers. In some cases, a combination of biodegradable and synthetic fibers is used to construct the RECP. Netting used to hold these fibers together is typically non-biodegradable as well.

- **Plastic netting** is a lightweight biaxially oriented netting designed for securing loose mulches like straw or paper to soil surfaces to establish vegetation. The netting is photodegradable. The netting is supplied in rolled strips, which must be secured with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Plastic mesh** is an open weave geotextile that is composed of an extruded synthetic fiber woven into a mesh with an opening size of less than ¼ in. It is used with re-vegetation or may be used to secure loose fiber such as straw to the ground. The material is supplied in rolled strips, which must be secured to the soil with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Synthetic fiber with netting** is a mat that is composed of durable synthetic fibers treated to resist chemicals and ultraviolet light. The mat is a dense, three dimensional mesh of synthetic (typically polyolefin) fibers stitched between two polypropylene nets. The mats are designed to be re-vegetated and provide a permanent composite system of soil, roots, and geomatrix. The material is furnished in rolled strips, which must be secured with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Bonded synthetic fibers** consist of a three dimensional geomatrix nylon (or other synthetic) matting. Typically it has more than 90 percent open area, which facilitates root growth. It's tough root reinforcing system anchors vegetation and protects against hydraulic lift and shear forces created by high volume discharges. It can be installed over prepared soil, followed by seeding into the mat. Once vegetated, it becomes an invisible composite system of soil, roots, and geomatrix. The material is furnished in rolled strips that must be secured with U-shaped staples or stakes in accordance with manufacturers' recommendations.

- **Combination synthetic and biodegradable RECPs** consist of biodegradable fibers, such as wood fiber or coconut fiber, with a heavy polypropylene net stitched to the top and a high strength continuous filament geomatrix or net stitched to the bottom. The material is designed to enhance re-vegetation. The material is furnished in rolled strips, which must be secured with U-shaped staples or stakes in accordance with manufacturers' recommendations.

Site Preparation

- Proper site preparation is essential to ensure complete contact of the blanket or matting with the soil.
- Grade and shape the area of installation.
- Remove all rocks, clods, vegetation or other obstructions so that the installed blankets or mats will have complete, direct contact with the soil.
- Prepare seedbed by loosening 2 to 3 in. of topsoil.

Seeding

Seed the area before blanket installation for erosion control and revegetation. Seeding after mat installation is often specified for turf reinforcement application. When seeding prior to blanket

installation, all check slots and other areas disturbed during installation must be re-seeded. Where soil filling is specified, seed the matting and the entire disturbed area after installation and prior to filling the mat with soil.

Fertilize and seed in accordance with seeding specifications or other types of landscaping plans. When using jute matting on a seeded area, apply approximately half the seed before laying the mat and the remainder after laying the mat. The protective matting can be laid over areas where grass has been planted and the seedlings have emerged. Where vines or other ground covers are to be planted, lay the protective matting first and then plant through matting according to design of planting.

Check Slots

Check slots are made of glass fiber strips, excelsior matting strips or tight folded jute matting blanket or strips for use on steep, highly erodible watercourses. The check slots are placed in narrow trenches 6 to 12 in. deep across the channel and left flush with the soil surface. They are to cover the full cross section of designed flow.

Laying and Securing Matting

- Before laying the matting, all check slots should be installed and the friable seedbed made free from clods, rocks, and roots. The surface should be compacted and finished according to the requirements of the manufacturer's recommendations.
- Mechanical or manual lay down equipment should be capable of handling full rolls of fabric and laying the fabric smoothly without wrinkles or folds. The equipment should meet the fabric manufacturer's recommendations or equivalent standards.

Anchoring

- U-shaped wire staples, metal geotextile stake pins, or triangular wooden stakes can be used to anchor mats and blankets to the ground surface.
- Wire staples should be made of minimum 11 gauge steel wire and should be U-shaped with 8 in. legs and 2 in. crown.
- Metal stake pins should be 0.188 in. diameter steel with a 1.5 in. steel washer at the head of the pin, and 8 in. in length.
- Wire staples and metal stakes should be driven flush to the soil surface.

Installation on Slopes

Installation should be in accordance with the manufacturer's recommendations. In general, these will be as follows:

- Begin at the top of the slope and anchor the blanket in a 6 in. deep by 6 in. wide trench. Backfill trench and tamp earth firmly.
- Unroll blanket down slope in the direction of water flow.
- Overlap the edges of adjacent parallel rolls 2 to 3 in. and staple every 3 ft.

- When blankets must be spliced, place blankets end over end (shingle style) with 6 in. overlap. Staple through overlapped area, approximately 12 in. apart.
- Lay blankets loosely and maintain direct contact with the soil. Do not stretch.
- Staple blankets sufficiently to anchor blanket and maintain contact with the soil. Staples should be placed down the center and staggered with the staples placed along the edges. Steep slopes, 1:1 (H:V) to 2:1 (H:V), require a minimum of 2 staples/yd². Moderate slopes, 2:1 (H:V) to 3:1 (H:V), require a minimum of 1 ½ staples/yd².

Installation in Channels

Installation should be in accordance with the manufacturer's recommendations. In general, these will be as follows:

- Dig initial anchor trench 12 in. deep and 6 in. wide across the channel at the lower end of the project area.
- Excavate intermittent check slots, 6 in. deep and 6 in. wide across the channel at 25 to 30 ft intervals along the channels.
- Cut longitudinal channel anchor trenches 4 in. deep and 4 in. wide along each side of the installation to bury edges of matting, whenever possible extend matting 2 to 3 in. above the crest of the channel side slopes.
- Beginning at the downstream end and in the center of the channel, place the initial end of the first roll in the anchor trench and secure with fastening devices at 12 in. intervals. Note: matting will initially be upside down in anchor trench.
- In the same manner, position adjacent rolls in anchor trench, overlapping the preceding roll a minimum of 3 in.
- Secure these initial ends of mats with anchors at 12 in. intervals, backfill and compact soil.
- Unroll center strip of matting upstream. Stop at next check slot or terminal anchor trench. Unroll adjacent mats upstream in similar fashion, maintaining a 3 in. overlap.
- Fold and secure all rolls of matting snugly into all transverse check slots. Lay mat in the bottom of the slot then fold back against itself. Anchor through both layers of mat at 12 in. intervals, then backfill and compact soil. Continue rolling all mat widths upstream to the next check slot or terminal anchor trench.
- Alternate method for non-critical installations: Place two rows of anchors on 6 in. centers at 25 to 30 ft. intervals in lieu of excavated check slots.
- Staple shingled lap spliced ends a minimum of 12 in. apart on 12 in. intervals.
- Place edges of outside mats in previously excavated longitudinal slots; anchor using prescribed staple pattern, backfill, and compact soil.
- Anchor, fill, and compact upstream end of mat in a 12 in. by 6 in. terminal trench.

- Secure mat to ground surface using U-shaped wire staples, geotextile pins, or wooden stakes.
- Seed and fill turf reinforcement matting with soil, if specified.

Soil Filling (if specified for turf reinforcement)

- Always consult the manufacturer's recommendations for installation.
- Do not drive tracked or heavy equipment over mat.
- Avoid any traffic over matting if loose or wet soil conditions exist.
- Use shovels, rakes, or brooms for fine grading and touch up.
- Smooth out soil filling just exposing top netting of mat.

Temporary Soil Stabilization Removal

- Temporary soil stabilization removed from the site of the work must be disposed of if necessary.

Costs

Relatively high compared to other BMPs. Biodegradable materials: $0.50 - $0.57/yd². Permanent materials: $3.00 - $4.50/yd². Staples: $0.04 - $0.05/staple. Approximate costs for installed materials are shown below:

Rolled Erosion Control Products		Installed Cost per Acre
Biodegradable	Jute Mesh	$6,500
	Curled Wood Fiber	$10,500
	Straw	$8,900
	Wood Fiber	$8,900
	Coconut Fiber	$13,000
	Coconut Fiber Mesh	$31,200
	Straw Coconut Fiber	$10,900
Non-Biodegradable	Plastic Netting	$2,000
	Plastic Mesh	$3,200
	Synthetic Fiber with Netting	$34,800
	Bonded Synthetic Fibers	$50,000
	Combination with Biodegradable	$32,000

Source: Caltrans Guidance for Soil Stabilization for Temporary Slopes, Nov. 1999

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season, and at two-week intervals during the non-rainy season.
- Inspect BMPs subject to non-stormwater discharges daily while non-stormwater discharges occur.

- Areas where erosion is evident shall be repaired and BMPs reapplied as soon as possible. Care should be exercised to minimize the damage to protected areas while making repairs, as any area damaged will require reapplication of BMPs.
- If washout or breakage occurs, re-install the material after repairing the damage to the slope or channel.
- Make sure matting is uniformly in contact with the soil.
- Check that all the lap joints are secure.
- Check that staples are flush with the ground.
- Check that disturbed areas are seeded.

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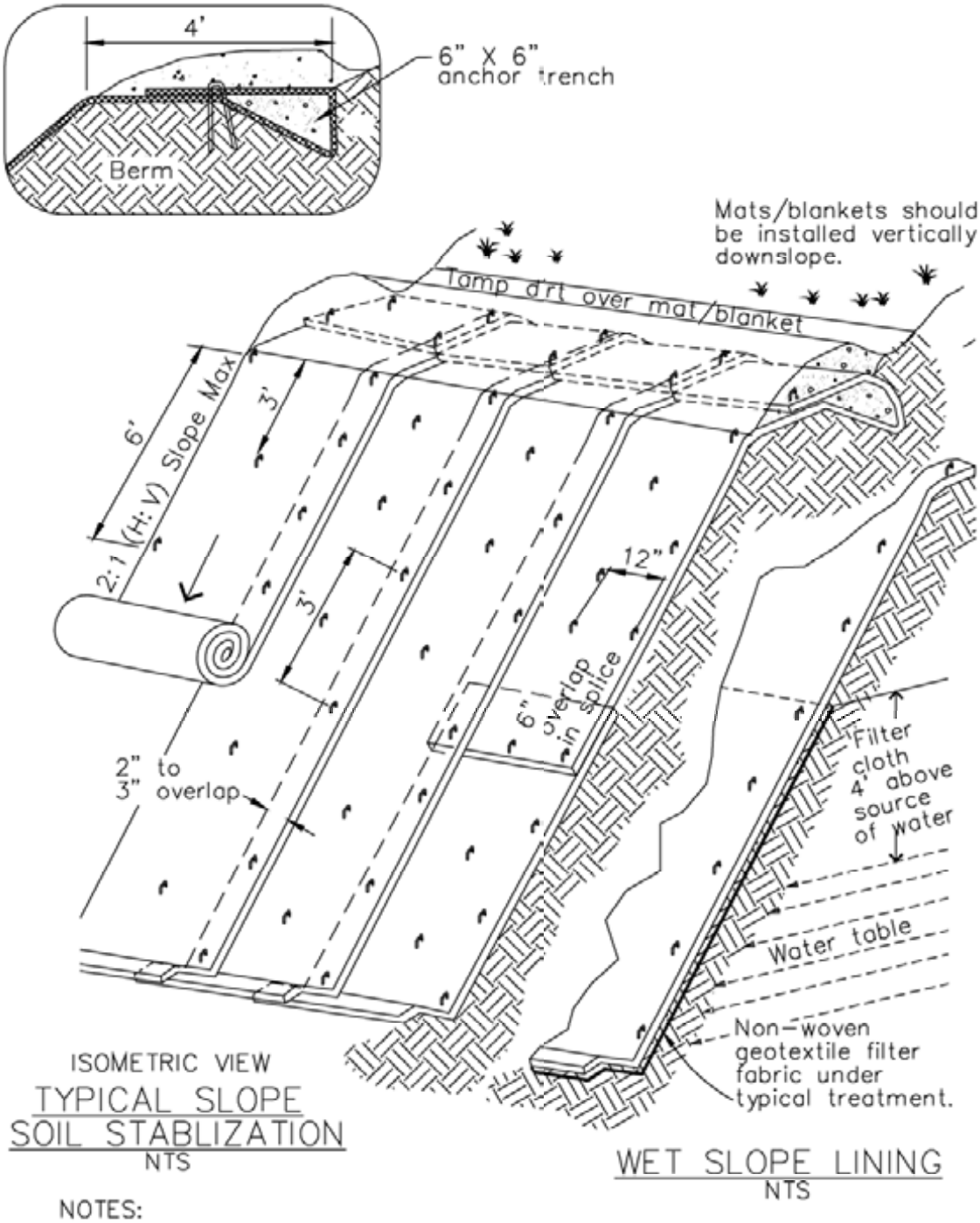


ISOMETRIC VIEW

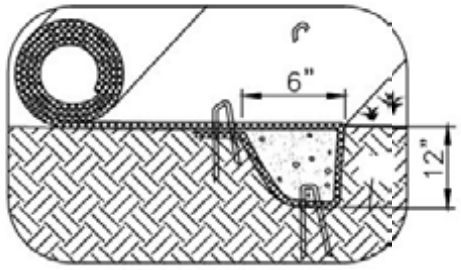
TYPICAL SLOPE
SOIL STABLIZATION
NTS

WET SLOPE LINING
NTS

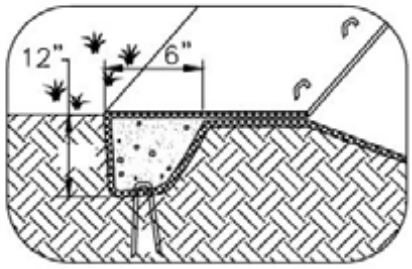
NOTES:

1. Slope surface shall be free of rocks, clods, sticks and grass. Mats/blankets shall have good soil contact.
2. Lay blankets loosely and stake or staple to maintain direct contact with the soil. Do not stretch.
3. Install per manufacturer's recommendations

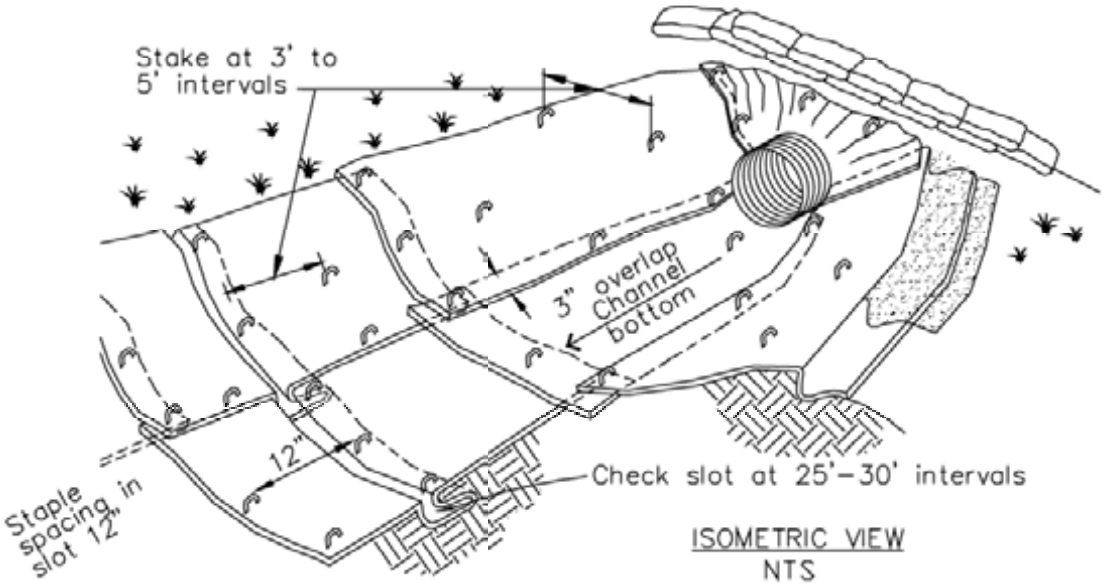
TYPICAL INSTALLATION DETAIL



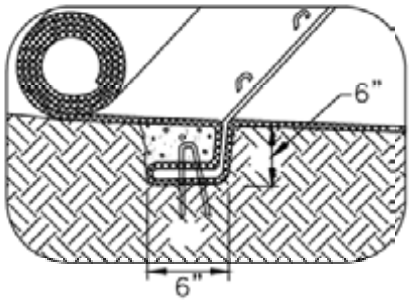
INITIAL CHANNEL ANCHOR TRENCH
NTS



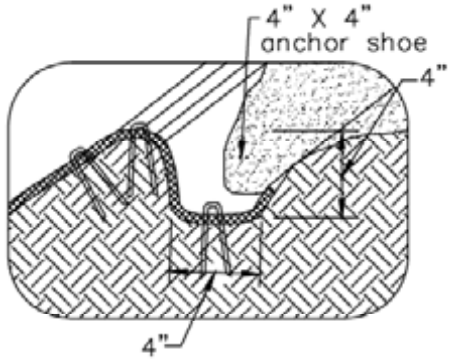
TERMINAL SLOPE AND CHANNEL ANCHOR TRENCH
NTS



ISOMETRIC VIEW
NTS



INTERMITTENT CHECK SLOT
NTS

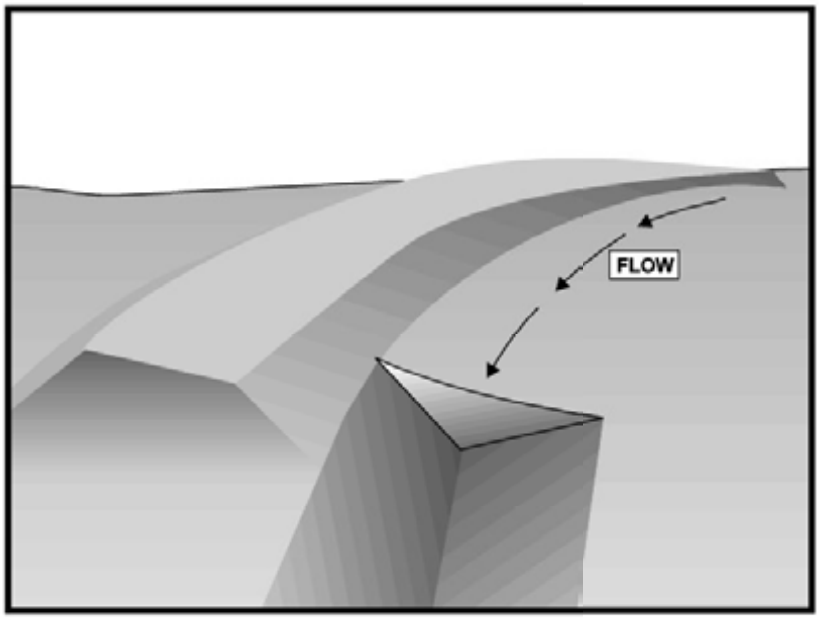


LONGITUDINAL ANCHOR TRENCH
NTS

NOTES:
1. Check slots to be constructed per manufacturers specifications.
2. Staking or stapling layout per manufacturers specifications.
3. Install per manufacturer's recommendations

TYPICAL INSTALLATION DETAIL

Earth Dikes and Drainage Swales EC-9



Objectives		
EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

An earth dike is a temporary berm or ridge of compacted soil used to divert runoff or channel water to a desired location. A drainage swale is a shaped and sloped depression in the soil surface used to convey runoff to a desired location. Earth dikes and drainage swales are used to divert off site runoff around the construction site, divert runoff from stabilized areas and disturbed areas, and direct runoff into sediment basins or traps.

Suitable Applications

Earth dikes and drainage swales are suitable for use, individually or together, where runoff needs to be diverted from one area and conveyed to another.

- Earth dikes and drainage swales may be used:
  - To convey surface runoff down sloping land
  - To intercept and divert runoff to avoid sheet flow over sloped surfaces
  - To divert and direct runoff towards a stabilized watercourse, drainage pipe or channel
  - To intercept runoff from paved surfaces
  - Below steep grades where runoff begins to concentrate
  - Along roadways and facility improvements subject to flood drainage

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- At the top of slopes to divert runon from adjacent or undisturbed slopes
- At bottom and mid slope locations to intercept sheet flow and convey concentrated flows
- Divert sediment laden runoff into sediment basins or traps

Limitations

Dikes should not be used for drainage areas greater than 10 acres or along slopes greater than 10 percent. For larger areas more permanent drainage structures should be built. All drainage structures should be built in compliance with local municipal requirements.

- Earth dikes may create more disturbed area on site and become barriers to construction equipment.
- Earth dikes must be stabilized immediately, which adds cost and maintenance concerns.
- Diverted stormwater may cause downstream flood damage.
- Dikes should not be constructed of soils that may be easily eroded.
- Regrading the site to remove the dike may add additional cost.
- Temporary drains and swales or any other diversion of runoff should not adversely impact upstream or downstream properties.
- Temporary drains and swales must conform to local floodplain management requirements.
- Earth dikes/drainage swales are not suitable as sediment trapping devices.
- It may be necessary to use other soil stabilization and sediment controls such as check dams, plastics, and blankets, to prevent scour and erosion in newly graded dikes, swales, and ditches.

Implementation

The temporary earth dike is a berm or ridge of compacted soil, located in such a manner as to divert stormwater to a sediment trapping device or a stabilized outlet, thereby reducing the potential for erosion and offsite sedimentation. Earth dikes can also be used to divert runoff from off site and from undisturbed areas away from disturbed areas and to divert sheet flows away from unprotected slopes.

An earth dike does not itself control erosion or remove sediment from runoff. A dike prevents erosion by directing runoff to an erosion control device such as a sediment trap or directing runoff away from an erodible area. Temporary diversion dikes should not adversely impact adjacent properties and must conform to local floodplain management regulations, and should not be used in areas with slopes steeper than 10%.

Slopes that are formed during cut and fill operations should be protected from erosion by runoff. A combination of a temporary drainage swale and an earth dike at the top of a slope can divert runoff to a location where it can be brought to the bottom of the slope (see EC-11, Slope Drains). A combination dike and swale is easily constructed by a single pass of a bulldozer or grader and

compacted by a second pass of the tracks or wheels over the ridge. Diversion structures should be installed when the site is initially graded and remain in place until post construction BMPs are installed and the slopes are stabilized.

Diversion practices concentrate surface runoff, increasing its velocity and erosive force. Thus, the flow out of the drain or swale must be directed onto a stabilized area or into a grade stabilization structure. If significant erosion will occur, a swale should be stabilized using vegetation, chemical treatment, rock rip-rap, matting, or other physical means of stabilization. Any drain or swale that conveys sediment laden runoff must be diverted into a sediment basin or trap before it is discharged from the site.

General

- Care must be applied to correctly size and locate earth dikes, drainage swales. Excessively steep, unlined dikes, and swales are subject to erosion and gully formation.
- Conveyances should be stabilized.
- Use a lined ditch for high flow velocities.
- Select flow velocity based on careful evaluation of the risks due to erosion of the measure, soil types, overtopping, flow backups, washout, and drainage flow patterns for each project site.
- Compact any fills to prevent unequal settlement.
- Do not divert runoff onto other property without securing written authorization from the property owner.
- When possible, install and utilize permanent dikes, swales, and ditches early in the construction process.
- Provide stabilized outlets.

Earth Dikes

Temporary earth dikes are a practical, inexpensive BMP used to divert stormwater runoff. Temporary diversion dikes should be installed in the following manner:

- All dikes should be compacted by earth moving equipment.
- All dikes should have positive drainage to an outlet.
- All dikes should have 2:1 or flatter side slopes, 18 in. minimum height, and a minimum top width of 24 in. Wide top widths and flat slopes are usually needed at crossings for construction traffic.
- The outlet from the earth dike must function with a minimum of erosion. Runoff should be conveyed to a sediment trapping device such as a Sediment Trap (SE-3) or Sediment Basin (SE-2) when either the dike channel or the drainage area above the dike are not adequately stabilized.

- Temporary stabilization may be achieved using seed and mulching for slopes less than 5% and either rip-rap or sod for slopes in excess of 5%. In either case, stabilization of the earth dike should be completed immediately after construction or prior to the first rain.
- If riprap is used to stabilize the channel formed along the toe of the dike, the following typical specifications apply:

Channel Grade	Riprap Stabilization
0.5-1.0%	4 in. Rock
1.1-2.0%	6 in. Rock
2.1-4.0%	8 in. Rock
4.1-5.0%	8 in. -12 in. Riprap

- The stone riprap, recycled concrete, etc. used for stabilization should be pressed into the soil with construction equipment.
- Filter cloth may be used to cover dikes in use for long periods.
- Construction activity on the earth dike should be kept to a minimum.

Drainage Swales

Drainage swales are only effective if they are properly installed. Swales are more effective than dikes because they tend to be more stable. The combination of a swale with a dike on the downhill side is the most cost effective diversion.

Standard engineering design criteria for small open channel and closed conveyance systems should be used (see the local drainage design manual). Unless local drainage design criteria state otherwise, drainage swales should be designed as follows:

- No more than 5 acres may drain to a temporary drainage swale.
- Place drainage swales above or below, not on, a cut or fill slope.
- Swale bottom width should be at least 2 ft
- Depth of the swale should be at least 18 in.
- Side slopes should be 2:1 or flatter.
- Drainage or swales should be laid at a grade of at least 1 percent, but not more than 15 percent.
- The swale must not be overtopped by the peak discharge from a 10-year storm, irrespective of the design criteria stated above.
- Remove all trees, stumps, obstructions, and other objectionable material from the swale when it is built.
- Compact any fill material along the path of the swale.

- Stabilize all swales immediately. Seed and mulch swales at a slope of less than 5 percent, and use rip-rap or sod for swales with a slope between 5 and 15 percent. For temporary swales, geotextiles and mats (EC-7) may provide immediate stabilization.
- Irrigation may be required to establish sufficient vegetation to prevent erosion.
- Do not operate construction vehicles across a swale unless a stabilized crossing is provided.
- Permanent drainage facilities must be designed by a professional engineer (see the local drainage design criteria for proper design).
- At a minimum, the drainage swale should conform to predevelopment drainage patterns and capacities.
- Construct the drainage swale with a positive grade to a stabilized outlet.
- Provide erosion protection or energy dissipation measures if the flow out of the drainage swale can reach an erosive velocity.

Costs

- Cost ranges from $15 to $55 per ft for both earthwork and stabilization and depends on availability of material, site location, and access.
- Small dikes: $2.50 - $6.50/linear ft; Large dikes: $2.50/yd^3.
- The cost of a drainage swale increases with drainage area and slope. Typical swales for controlling internal erosion are inexpensive, as they are quickly formed during routine earthwork.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Inspect BMPs subject to non-stormwater discharges daily while non-stormwater discharges occur.
- Inspect ditches and berms for washouts. Replace lost riprap, damaged linings or soil stabilizers as needed.
- Inspect channel linings, embankments, and beds of ditches and berms for erosion and accumulation of debris and sediment. Remove debris and sediment and repair linings and embankments as needed.
- Temporary conveyances should be completely removed as soon as the surrounding drainage area has been stabilized or at the completion of construction

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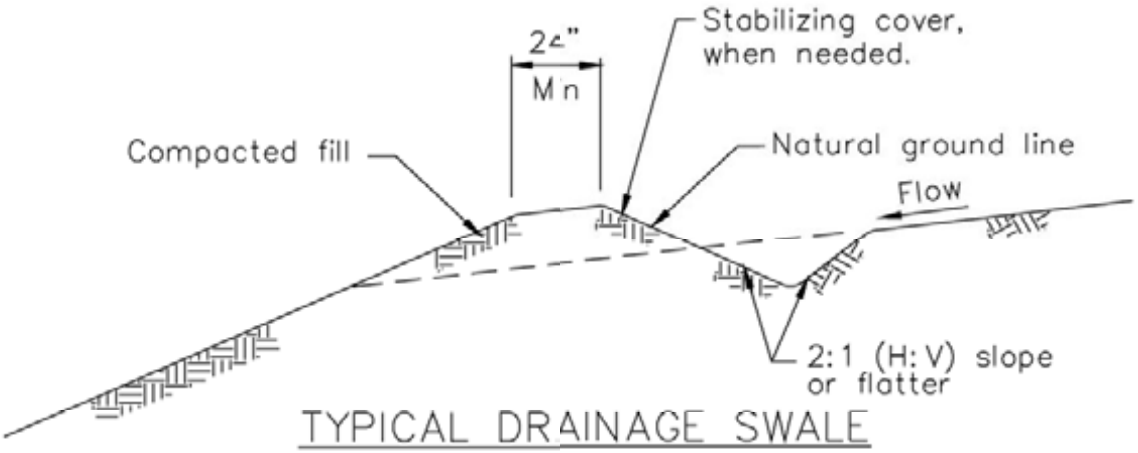
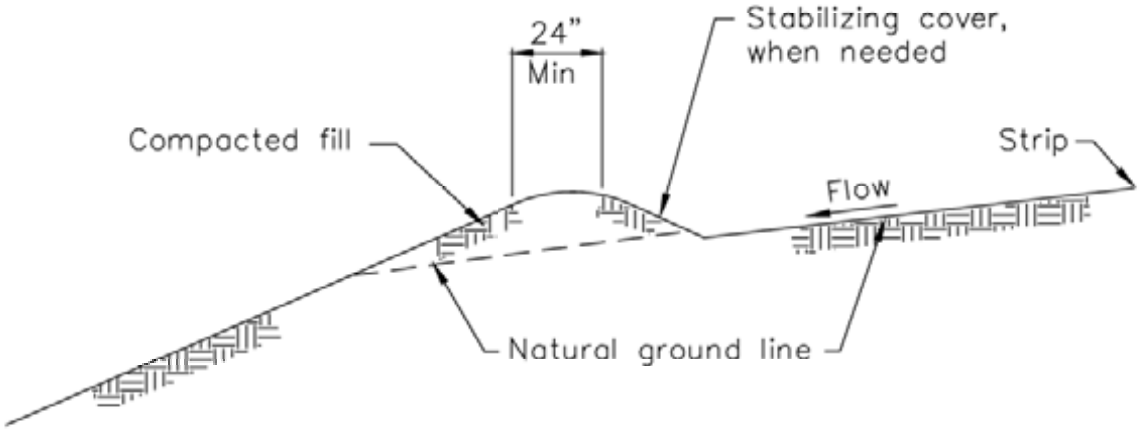


TYPICAL DRAINAGE SWALE
NOT TO SCALE

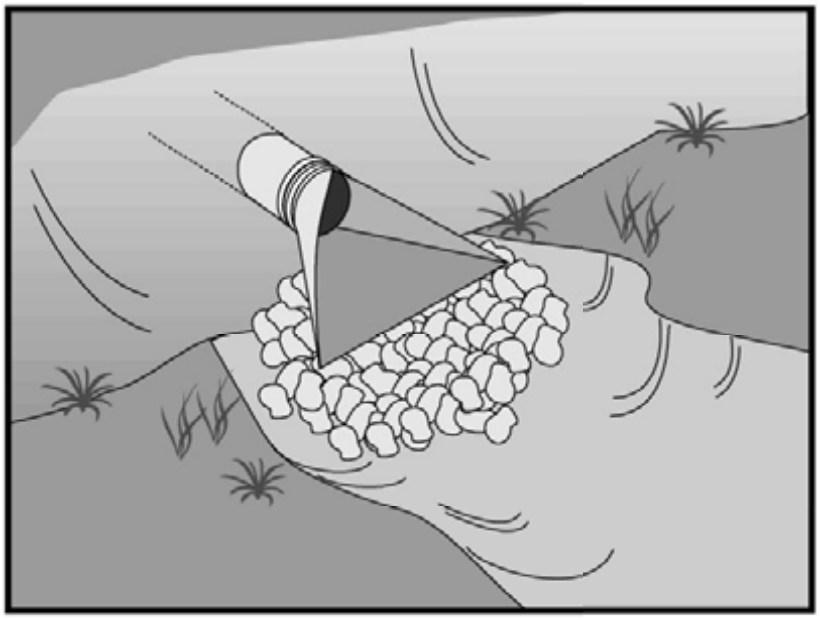
NOTES:

1. Stabilize inlet, outlets and slopes.
2. Properly compact the subgrade.



TYPICAL EARTH DIKE
NOT TO SCALE

Velocity Dissipation Devices EC-10



Objectives		
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WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

Outlet protection is a physical device composed of rock, grouted riprap, or concrete rubble, which is placed at the outlet of a pipe or channel to prevent scour of the soil caused by concentrated, high velocity flows.

Suitable Applications

Whenever discharge velocities and energies at the outlets of culverts, conduits, or channels are sufficient to erode the next downstream reach. This includes temporary diversion structures to divert runon during construction.

- These devices may be used at the following locations:
  - Outlets of pipes, drains, culverts, slope drains, diversion ditches, swales, conduits, or channels.
  - Outlets located at the bottom of mild to steep slopes.
  - Discharge outlets that carry continuous flows of water.
  - Outlets subject to short, intense flows of water, such as flash floods.
  - Points where lined conveyances discharge to unlined conveyances

Limitations

- Large storms or high flows can wash away the rock outlet protection and leave the area susceptible to erosion.

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- Sediment captured by the rock outlet protection may be difficult to remove without removing the rock.
- Outlet protection may negatively impact the channel habitat.
- Grouted riprap may break up in areas of freeze and thaw.
- If there is not adequate drainage, and water builds up behind grouted riprap, it may cause the grouted riprap to break up due to the resulting hydrostatic pressure.

Implementation

General

Outlet protection is needed where discharge velocities and energies at the outlets of culverts, conduits or channels are sufficient to erode the immediate downstream reach. This practice protects the outlet from developing small eroded pools (plange pools), and protects against gully erosion resulting from scouring at a culvert mouth.

Design and Layout

As with most channel design projects, depth of flow, roughness, gradient, side slopes, discharge rate, and velocity should be considered in the outlet design. Compliance to local and state regulations should also be considered while working in environmentally sensitive streambeds. General recommendations for rock size and length of outlet protection mat are shown in the rock outlet protection figure in this BMP and should be considered minimums. The apron length and rock size gradation are determined using a combination of the discharge pipe diameter and estimate discharge rate: Select the longest apron length and largest rock size suggested by the pipe size and discharge rate. Where flows are conveyed in open channels such as ditches and swales, use the estimated discharge rate for selecting the apron length and rock size. Flows should be same as the culvert or channel design flow but never the less than the peak 5 year flow for temporary structures planned for one rainy season, or the 10 year peak flow for temporary structures planned for two or three rainy seasons.

- There are many types of energy dissipaters, with rock being the one that is represented in the attached figure.
- Best results are obtained when sound, durable, and angular rock is used.
- Install riprap, grouted riprap, or concrete apron at selected outlet. Riprap aprons are best suited for temporary use during construction. Grouted or wired tied rock riprap can minimize maintenance requirements.
- Rock outlet protection is usually less expensive and easier to install than concrete aprons or energy dissipaters. It also serves to trap sediment and reduce flow velocities.
- Carefully place riprap to avoid damaging the filter fabric.
    - Stone 4 in. to 6 in. may be carefully dumped onto filter fabric from a height not to exceed 12 in.
    - Stone 8 in. to 12 in. must be hand placed onto filter fabric, or the filter fabric may be covered with 4 in. of gravel and the 8 in. to 12 in. rock may be dumped from a height not to exceed 16 in.

- Stone greater than 12 in. shall only be dumped onto filter fabric protected with a layer of gravel with a thickness equal to one half the D_{50} rock size, and the dump height limited to twice the depth of the gravel protection layer thickness.

- For proper operation of apron: Align apron with receiving stream and keep straight throughout its length. If a curve is needed to fit site conditions, place it in upper section of apron.

- Outlets on slopes steeper than 10 percent should have additional protection.

Costs

Costs are low if material is readily available. If material is imported, costs will be higher. Average installed cost is $150 per device.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.

- Inspect BMPs subjected to non-stormwater discharges daily while non-stormwater discharges occur.

- Inspect apron for displacement of the riprap and damage to the underlying fabric. Repair fabric and replace riprap that has washed away. If riprap continues to wash away, consider using larger material.

- Inspect for scour beneath the riprap and around the outlet. Repair damage to slopes or underlying filter fabric immediately.

- Temporary devices should be completely removed as soon as the surrounding drainage area has been stabilized or at the completion of construction.

References

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Erosion and Sediment Control Handbook, S.J. Goldman, K. Jackson, T.A. Bursztynsky, P.E., McGraw Hill Book Company, 1986.

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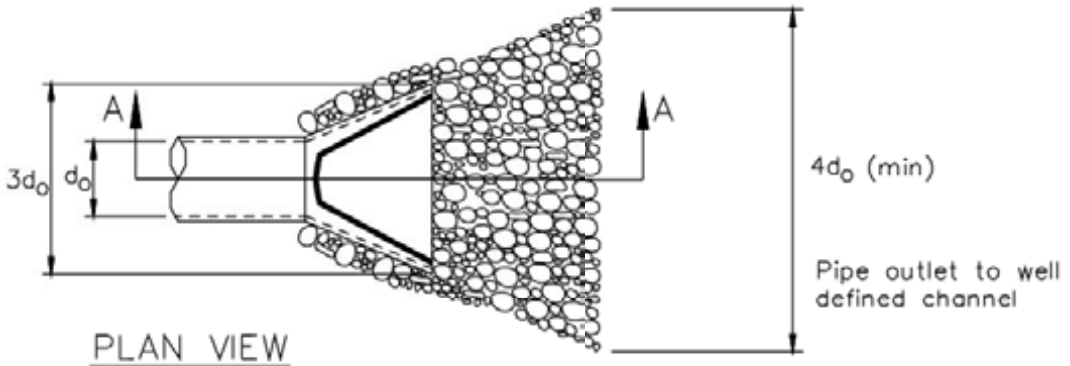
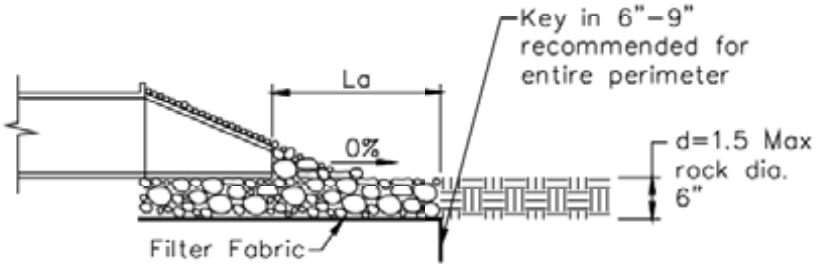


PLAN VIEW

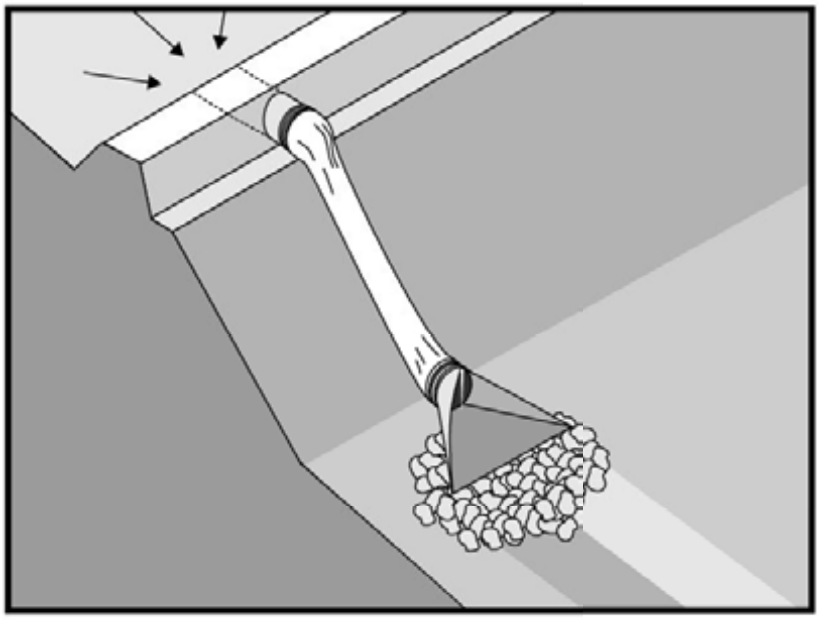


SECTION A–A

Pipe Diameter inches	Discharge ft³/s	Apron Length, La ft	Rip Rap D_{50} Diameter Min inches
12	5	10	4
	10	13	6
18	10	10	6
	20	16	8
	30	23	12
	40	26	16
24	30	16	8
	40	26	8
	50	26	12
	60	30	16

For larger or higher flows consult a Registered Civil Engineer
Source: USDA - SCS

Slope Drains EC-11



Objectives		
EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

EC-9 Earth Dike, Drainage Swales

Description and Purpose

A slope drain is a pipe used to intercept and direct surface runoff or groundwater into a stabilized watercourse, trapping device, or stabilized area. Slope drains are used with earth dikes and drainage ditches to intercept and direct surface flow away from slope areas to protect cut or fill slopes.

Suitable Applications

- Where concentrated flow of surface runoff must be conveyed down a slope in order to prevent erosion.
- Drainage for top of slope diversion dikes or swales.
- Drainage for top of cut and fill slopes where water can accumulate.
- Emergency spillway for a sediment basin.

Limitations

Installation is critical for effective use of the pipe slope drain to minimize potential gully erosion.

- Maximum drainage area per slope drain is 10 acres. (For large areas use a paved chute, rock lined channel, or additional pipes.)
- Severe erosion may result when slope drains fail by overtopping, piping, or pipe separation.

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  - During large storms, pipe slope drains may become clogged or over charged, forcing water around the pipe and causing extreme slope erosion.
  - If the sectional downdrain is not sized correctly, the runoff can spill over the drain sides causing gully erosion and potential failure of the structure.

- Dissipation of high flow velocities at the pipe outlet is required to avoid downstream erosion.

Implementation

General

The slope drain is applicable for any construction site where concentrated surface runoff can accumulate and must be conveyed down the slope in order to prevent erosion. The slope drain is effective because it prevents the stormwater from flowing directly down the slope by confining all the runoff into an enclosed pipe or channel. Due to the time lag between grading slopes and installation of permanent stormwater collection systems and slope stabilization measures, temporary provisions to intercept runoff are sometimes necessary. Particularly in steep terrain, slope drains can protect unstabilized areas from erosion.

Installation

The slope drain may be a rigid pipe, such as corrugated metal, a flexible conduit, or a lined terrace drain with the inlet placed on the top of a slope and the outlet at the bottom of the slope. This BMP typically is used in combination with a diversion control, such as an earth dike or drainage swale at the top of the slope.

The following criteria must be considered when siting slope drains.

- Permanent structures included in the project plans can often serve as construction BMPs if implemented early. However, the permanent structure must meet or exceed the criteria for the temporary structure.
- Inlet structures must be securely entrenched and compacted to avoid severe gully erosion.
- Slope drains must be securely anchored to the slope and must be adequately sized to carry the capacity of the design storm and associated forces.
- Outlets must be stabilized with riprap, concrete or other type of energy dissipator, or directed into a stable sediment trap or basin. See EC-10, Velocity Dissipation Devices.
- Debris racks are recommended at the inlet. Debris racks located several feet upstream of the inlet can usually be larger than racks at the inlet, and thus provide enhanced debris protection and less plugging.
- Safety racks are also recommended at the inlet and outlet of pipes where children or animals could become entrapped.
- Secure inlet and surround with dikes to prevent gully erosion and anchor pipe to slope.
- When using slope drains, limit drainage area to 10 acres per pipe. For larger areas, use a rock lined channel or a series of pipes.

- Size to convey at least the peak flow of a 10-year storm. The design storm is conservative due to the potential impact of system failures.
- Maximum slope generally limited to 2:1 (H:V) as energy dissipation below steeper slopes is difficult.
- Direct surface runoff to slope drains with interceptor dikes. See BMP EC-9, Earth Dikes and Drainage Swales. Top of interceptor dikes should be 12 in. higher than the top of the slope drain.
- Slope drains can be placed on or buried underneath the slope surface.
- Recommended materials include both metal and plastic pipe, either corrugated or smooth wall. Concrete pipe can also be used.
- When installing slope drains:
  - Install slope drains perpendicular to slope contours.
  - Compact soil around and under entrance, outlet, and along length of pipe.
  - Securely anchor and stabilize pipe and appurtenances into soil.
  - Check to ensure that pipe connections are watertight.
  - Protect area around inlet with filter cloth. Protect outlet with riprap or other energy dissipation device. For high energy discharges, reinforce riprap with concrete or use reinforced concrete device.
  - Protect outlet of slope drains using a flared end section when outlet discharges to a flexible energy dissipation device.
  - A flared end section installed at the inlet will improve flow into the slope drain and prevent erosion at the pipe entrance. Use a flared end section with a 6 in. minimum toe plate to help prevent undercutting. The flared section should slope towards the pipe inlet.

Design and Layout

The capacity for temporary drains should be sufficient to convey at least the peak runoff from a 10-year rainfall event. The pipe size may be computed using the Rational Method or a method established by the local municipality. Higher flows must be safely stored or routed to prevent any offsite concentration of flow and any erosion of the slope. The design storm is purposely conservative due to the potential impacts associated with system failures.

As a guide, temporary pipe slope drains should not be sized smaller than shown in the following table:

Minimum Pipe Diameter (Inches)	Maximum Drainage Area (Acres)
12	1.0
18	3.0
21	5.0
24	7.0
30	10.0

Larger drainage areas can be treated if the area can be subdivided into areas of 10 acres or less and each area is treated as a separate drainage. Drainage areas exceeding 10 acres must be designed by a Registered Civil Engineer and approved by the agency that issued the grading permit.

Materials:

Soil type, rainfall patterns, construction schedule, local requirements, and available supply are some of the factors to be considered when selecting materials. The following types of slope drains are commonly used:

- **Rigid Pipe:** This type of slope drain is also known as a pipe drop. The pipe usually consists of corrugated metal pipe or rigid plastic pipe. The pipe is placed on undisturbed or compacted soil and secured onto the slope surface or buried in a trench. Concrete thrust blocks must be used when warranted by the calculated thrust forces. Collars should be properly installed and secured with metal strappings or watertight collars.

- **Flexible Pipe**: The flexible pipe slope drain consists of a flexible tube of heavy-duty plastic, rubber, or composite material. The tube material is securely anchored onto the slope surface. The tube should be securely fastened to the metal inlet and outlet conduit sections with metal strappings or watertight collars.

- **Section Downdrains**: The section downdrain consists of pre-fabricated, section conduit of half round or third round material. The sectional downdrain performs similar to a flume or chute. The pipe must be placed on undisturbed or compacted soil and secured into the slope.

- **Concrete-lined Terrace Drain:** This is a concrete channel for draining water from a terrace on a slope to the next level. These drains are typically specified as permanent structures and, if installed early, can serve as slope drains during construction, which should be designed according to local drainage design criteria.

Costs

- Cost varies based on pipe selection and selected outlet protection.

Corrugated Steel Pipes, Per Foot	
Size	Supplied and Installed Cost (No Trenching Included)
12"	$19.60 per LF
15"	$22.00
18"	$26.00
24"	$32.00
30"	$50.00
PVC Pipes, Per Foot	
Size	Supplied and Installed Cost (No Trenching Included)
12"	$24.50
14"	$49.00
16"	$51.00
18"	$54.00
20"	$66.00
24"	$93.00
30"	$130.00

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Inspect BMPs subjected to non-stormwater discharges daily while non-stormwater discharges occur.
- Inspect outlet for erosion and downstream scour. If eroded, repair damage and install additional energy dissipation measures. If downstream scour is occurring, it may be necessary to reduce flows being discharged into the channel unless other preventative measures are implemented.
- Insert inlet for clogging or undercutting. Remove debris from inlet to maintain flows. Repair undercutting at inlet and if needed, install flared section or rip rap around the inlet to prevent further undercutting.
- Inspect pipes for leakage. Repair leaks and restore damaged slopes.
- Inspect slope drainage for accumulations of debris and sediment.
- Remove built up sediment from entrances and outlets as required. Flush drains if necessary; capture and settle out sediment from discharge.

- Make sure water is not ponding onto inappropriate areas (e.g., active traffic lanes, material storage areas, etc.).
- Pipe anchors must be checked to ensure that the pipe remains anchored to the slope. Install additional anchors if pipe movement is detected.

References

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National Association of Home Builders (NAHB). Stormwater Runoff & Nonpoint Source Pollution Control Guide for Builders and Developers. National Association of Home Builders, Washington, D.C., 1995

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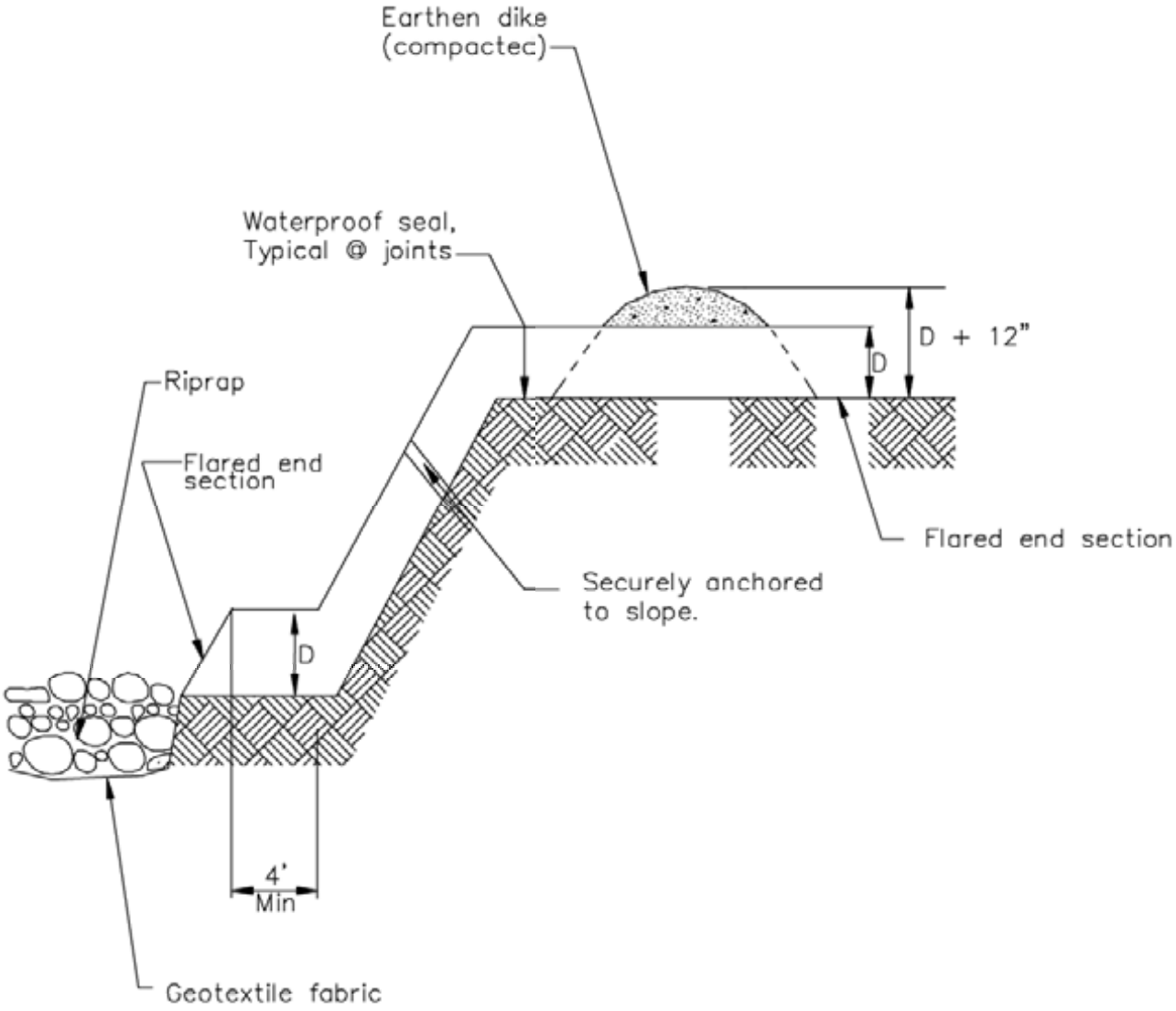
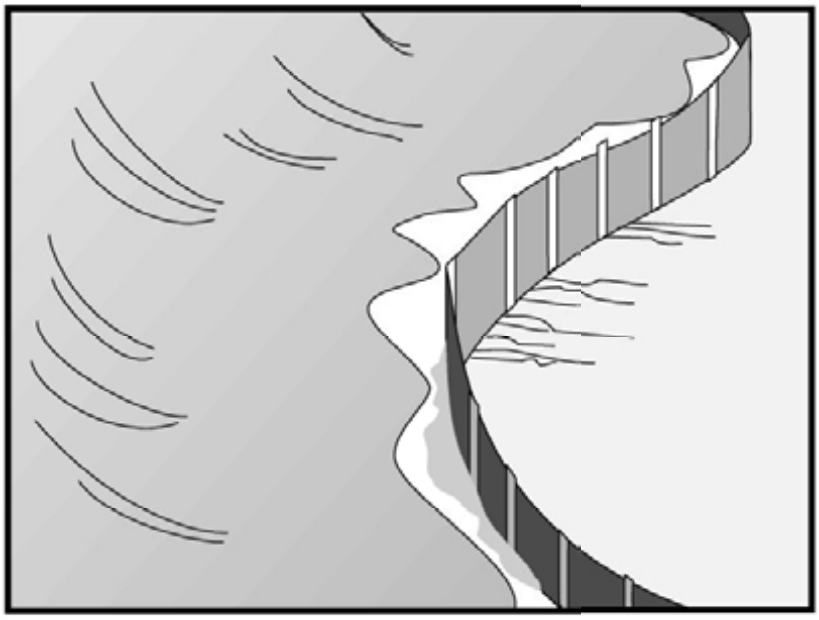


TYPICAL SLOPE DRAIN
NOT TO SCALE

Silt Fence SE-1



Objectives

EC	Erosion Control	☑
SE	Sediment Control	
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-8 Sandbag Barrier

SE-9 Straw Bale Barrier

Description and Purpose

A silt fence is made of a filter fabric that has been entrenched, attached to supporting poles, and sometimes backed by a plastic or wire mesh for support. The silt fence detains sediment-laden water, promoting sedimentation behind the fence.

Suitable Applications

Silt fences are suitable for perimeter control, placed below areas where sheet flows discharge from the site. They should also be used as interior controls below disturbed areas where runoff may occur in the form of sheet and rill erosion. Silt fences are generally ineffective in locations where the flow is concentrated and are only applicable for sheet or overland flows. Silt fences are most effective when used in combination with erosion controls. Suitable applications include:

- Along the perimeter of a project.
- Below the toe or down slope of exposed and erodible slopes.
- Along streams and channels.
- Around temporary spoil areas and stockpiles.
- Below other small cleared areas.

Limitations

- Do not use in streams, channels, drain inlets, or anywhere flow is concentrated.

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- Do not use in locations where ponded water may cause flooding.
- Do not place fence on a slope, or across any contour line. If not installed at the same elevation throughout, silt fences will create erosion.
- Filter fences will create a temporary sedimentation pond on the upstream side of the fence and may cause temporary flooding. Fences not constructed on a level contour will be overtopped by concentrated flow resulting in failure of the filter fence.
- Improperly installed fences are subject to failure from undercutting, overlapping, or collapsing.
  - Not effective unless trenched and keyed in.
  - Not intended for use as mid-slope protection on slopes greater than 4:1 (H:V).
  - Do not allow water depth to exceed 1.5 ft at any point.

Implementation

General

A silt fence is a temporary sediment barrier consisting of filter fabric stretched across and attached to supporting posts, entrenched, and, depending upon the strength of fabric used, supported with plastic or wire mesh fence. Silt fences trap sediment by intercepting and detaining small amounts of sediment-laden runoff from disturbed areas in order to promote sedimentation behind the fence.

Silt fences are preferable to straw bale barriers in many cases. Laboratory work at the Virginia Highway and Transportation Research Council has shown that silt fences can trap a much higher percentage of suspended sediments than can straw bales. While the failure rate of silt fences is lower than that of straw bale barriers, there are many instances where silt fences have been improperly installed. The following layout and installation guidance can improve performance and should be followed:

- Use principally in areas where sheet flow occurs
- Don't use in streams, channels, or anywhere flow is concentrated. Don't use silt fences to divert flow.
- Don't use below slopes subject to creep, slumping, or landslides.
- Select filter fabric that retains 85% of soil by weight, based on sieve analysis, but that is not finer than an equivalent opening size of 70.
- Install along a level contour, so water does not pond more than 1.5 ft at any point along the silt fence.
- The maximum length of slope draining to any point along the silt fence should be 200 ft or less.
- The maximum slope perpendicular to the fence line should be 1:1.

- Provide sufficient room for runoff to pond behind the fence and to allow sediment removal equipment to pass between the silt fence and toes of slopes or other obstructions. About 1200 ft² of ponding area should be provided for every acre draining to the fence.
- Turn the ends of the filter fence uphill to prevent stormwater from flowing around the fence.
- Leave an undisturbed or stabilized area immediately down slope from the fence where feasible.
- Silt fences should remain in place until the disturbed area is permanently stabilized.

Design and Layout

Selection of a filter fabric is based on soil conditions at the construction site (which affect the equivalent opening size (EOS) fabric specification) and characteristics of the support fence (which affect the choice of tensile strength). The designer should specify a filter fabric that retains the soil found on the construction site yet that it has openings large enough to permit drainage and prevent clogging. The following criteria is recommended for selection of the equivalent opening size:

1. If 50 percent or less of the soil, by weight, will pass the U.S. Standard Sieve No. 200, select the EOS to retain 85 % of the soil. The EOS should not be finer than EOS 70.
2. For all other soil types, the EOS should be no larger than the openings in the U.S. Standard Sieve No. 70 except where direct discharge to a stream, lake, or wetland will occur, then the EOS should be no larger than Standard Sieve No. 100.

To reduce the chance of clogging, it is preferable to specify a fabric with openings as large as allowed by the criteria. No fabric should be specified with an EOS smaller than U.S. Standard Sieve No. 100. If 85% or more of a soil, by weight, passes through the openings in a No. 200 sieve, filter fabric should not be used. Most of the particles in such a soil would not be retained if the EOS was too large and they would clog the fabric quickly if the EOS were small enough to capture the soil.

The fence should be supported by a plastic or wire mesh if the fabric selected does not have sufficient strength and bursting strength characteristics for the planned application (as recommended by the fabric manufacturer). Filter fabric material should contain ultraviolet inhibitors and stabilizers to provide a minimum of six months of expected usable construction life at a temperature range of 0 °F to 120 °F.

- Layout in accordance with attached figures.
- For slopes steeper than 2:1 (H:V) and that contain a high number of rocks or large dirt clods that tend to dislodge, it may be necessary to install additional protection immediately adjacent to the bottom of the slope, prior to installing silt fence. Additional protection may be a chain link fence or a cable fence.
- For slopes adjacent to sensitive receiving waters or Environmentally Sensitive Areas (ESAs), silt fence should be used in conjunction with erosion control BMPs.

Materials

- Silt fence fabric should be woven polypropylene with a minimum width of 36 in. and a minimum tensile strength of 100 lb force. The fabric should conform to the requirements in ASTM designation D4632 and should have an integral reinforcement layer. The reinforcement layer should be a polypropylene, or equivalent, net provided by the manufacturer. The permittivity of the fabric should be between 0.1 sec^{-1} and 0.15 sec^{-1} in conformance with the requirements in ASTM designation D4491.

- Wood stakes should be commercial quality lumber of the size and shape shown on the plans. Each stake should be free from decay, splits or cracks longer than the thickness of the stake or other defects that would weaken the stakes and cause the stakes to be structurally unsuitable.

- Staples used to fasten the fence fabric to the stakes should be not less than 1.75 in. long and should be fabricated from 15 gauge or heavier wire. The wire used to fasten the tops of the stakes together when joining two sections of fence should be 9 gauge or heavier wire. Galvanizing of the fastening wire will not be required.

- There are new products that may use prefabricated plastic holders for the silt fence and use bar reinforcement instead of wood stakes. If bar reinforcement is used in lieu of wood stakes, use number four or greater bar. Provide end protection for any exposed bar reinforcement.

Installation Guidelines

Silt fences are to be constructed on a level contour. Sufficient area should exist behind the fence for ponding to occur without flooding or overtopping the fence.

- A trench should be excavated approximately 6 in. wide and 6 in. deep along the line the proposed silt fence.

- Bottom of the silt fence should be keyed-in a minimum of 12 in.

- Posts should be spaced a maximum of 6 ft apart and driven securely into the ground a minimum of 18 in. or 12 in. below the bottom of the trench.

- When standard strength filter fabric is used, a plastic or wire mesh support fence should be fastened securely to the upslope side of posts using heavy–duty wire staples at least 1 in. long. The mesh should extend into the trench. When extra-strength filter fabric and closer post spacing are used, the mesh support fence may be eliminated. Filter fabric should be purchased in a long roll, then cut to the length of the barrier. When joints are necessary, filter cloth should be spliced together only at a support post, with a minimum 6 in. overlap and both ends securely fastened to the post.

- The trench should be backfilled with compacted native material.

- Construct silt fences with a setback of at least 3 ft from the toe of a slope. Where a silt fence is determined to be not practicable due to specific site conditions, the silt fence may be constructed at the toe of the slope, but should be constructed as far from the toe of the slope as practicable. Silt fences close to the toe of the slope will be less effective and difficult to maintain.

- Construct the length of each reach so that the change in base elevation along the reach does not exceed 1/3 the height of the barrier; in no case should the reach exceed 500 ft.

Costs

- Average annual cost for installation and maintenance (assumes 6 month useful life): $7 per lineal foot ($850 per drainage acre). Range of cost is $3.50 - $9.10 per lineal foot.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Repair undercut silt fences.
- Repair or replace split, torn, slumping, or weathered fabric. The lifespan of silt fence fabric is generally 5 to 8 months.
- Silt fences that are damaged and become unsuitable for the intended purpose should be removed from the site of work, disposed of, and replaced with new silt fence barriers.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed at an appropriate location.
- Silt fences should be left in place until the upstream area is permanently stabilized. Until then, the silt fence must be inspected and maintained.
- Holes, depressions, or other ground disturbance caused by the removal of the silt fences should be backfilled and repaired.

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Proposed Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters, Work Group-Working Paper, USEPA, April 1992.

Sedimentation and Erosion Control Practices, and Inventory of Current Practices (Draft), UESPA, 1990.

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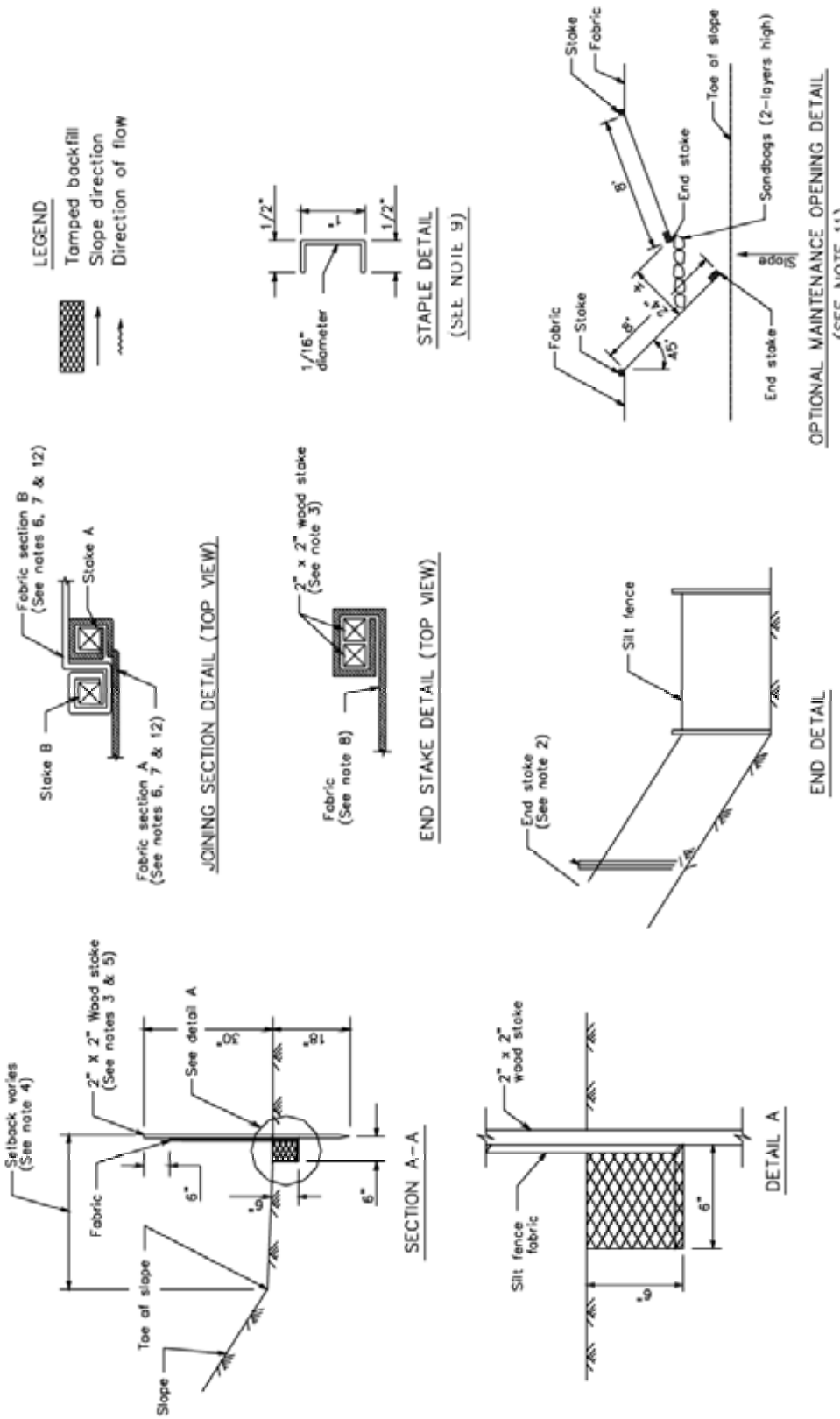
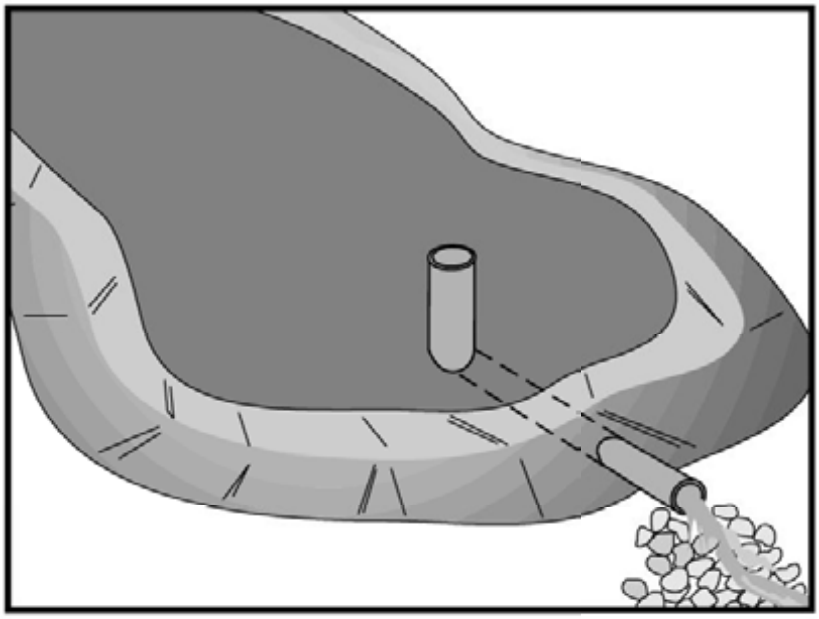
Water Quality Management Plan for the Lake Tahoe Region, Volume II, Handbook of Management Practices, Tahoe Regional Planning Agency, November 1988.

NOTES

1. Construct the length of each reach so that the change in base elevation along the reach does not exceed 1/3 the height of the linear barrier, in no case shall the reach length exceed 500'.
2. The last 8'-0" of fence shall be turned up slope.
3. Stake dimensions are nominal.
4. Dimension may vary to fit field condition.
5. Stakes shall be spaced at 8'-0" maximum and shall be positioned on downstream side of fence.
6. Stakes to overlap and fence fabric to fold around each stake one full turn. Secure fabric to stake with 4 staples.
7. Stakes shall be driven tightly together to prevent potential flow-through of sediment at joint. The tops of the stakes shall be secured with wire.
8. For end stake, fence fabric shall be folded around two stakes one full turn and secured with 4 staples.
9. Minimum 4 staples per stake. Dimensions shown are typical.
10. Cross barriers shall be a minimum of 1/3 and a maximum of 1/2 the height of the linear barrier.
11. Maintenance openings shall be constructed in a manner to ensure sediment remains behind silt fence.
12. Joining sections shall not be placed at sump locations.
13. Sandbag rows and layers shall be offset to eliminate gaps.


LEGEND
Tamped backfill
Slope direction
Direction of flow
1/2"
1"
1/2"
1/16" diameter
STAPLE DETAIL
(SEE NOTE 9)
Stake
Fabric
Toe of slope
8'
End stake
Sandbags (2-layers high)
Slope
Fabric
Stake
8"
4"
45°
End stake
OPTIONAL MAINTENANCE OPENING DETAIL
(SEE NOTE 11)
Fabric section B
(See notes 6, 7 & 12)
Stake A
Stake B
Fabric section A
(See notes 6, 7 & 12)
JOINING SECTION DETAIL (TOP VIEW)
2" x 2" wood stake
(See note 3)
Fabric
(See note 8)
END STAKE DETAIL (TOP VIEW)
Silt fence
End stake
(See note 2)
END DETAIL
Setback varies
(See note 4)
2" X 2" Wood stake
(See notes 3 & 5)
See detail A
10"
10"
Fabric
6"
6"
Toe of slope
Slope
SECTION A-A
2" x 2"
wood stake
Silt fence
fabric
6"
DETAIL A

Sediment Basin SE-2



Objectives		
EC	Erosion Control	
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-3 Sediment Trap (for smaller areas)

Description and Purpose

A sediment basin is a temporary basin formed by excavation or by constructing an embankment so that sediment-laden runoff is temporarily detained under quiescent conditions, allowing sediment to settle out before the runoff is discharged.

Suitable Applications

Sediment basins may be suitable for use on larger projects with sufficient space for constructing the basin. Sediment basins should be considered for use:

- Where sediment-laden water may enter the drainage system or watercourses
- On construction projects with disturbed areas during the rainy season
- At the outlet of disturbed watersheds between 5 acres and 75 acres
- At the outlet of large disturbed watersheds, as necessary
- Where post construction detention basins are required
- In association with dikes, temporary channels, and pipes used to convey runoff from disturbed areas

Limitations

Sediment basins must be installed only within the property limits and where failure of the structure will not result in loss of life, damage to homes or buildings, or interruption of use or service of

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public roads or utilities. In addition, sediment basins are attractive to children and can be very dangerous. Local ordinances regarding health and safety must be adhered to. If fencing of the basin is required, the type of fence and its location should be shown in the SWPPP and in the construction specifications.

- Generally, sediment basins are limited to drainage areas of 5 acres or more, but not appropriate for drainage areas greater than 75 acres.
- Sediment basins may become an "attractive nuisance" and care must be taken to adhere to all safety practices. If safety is a concern, basin may require protective fencing.
- Sediment basins designed according to this handbook are only practically effective in removing sediment down to about the medium silt size fraction. Sediment-laden runoff with smaller size fractions (fine silt and clay) may not be adequately treated unless chemical treatment is used in addition to the sediment basin.
- Sites with very fine sediments (fine silt and clay) may require longer detention times for effective sediment removal.
- Basins with a height of 25 ft or more or an impounding capacity of 50 ac-ft or more must obtain approval from Division of Safety of Dams.
- Standing water may cause mosquitoes or other pests to breed.
- Basins require large surface areas to permit settling of sediment. Size may be limited by the available area.

Implementation

General

A sediment basin is a controlled stormwater release structure formed by excavation or by construction of an embankment of compacted soil across a drainage way, or other suitable location. It is intended to trap sediment before it leaves the construction site. The basin is a temporary measure with a design life of 12 to 28 months in most cases and is to be maintained until the site area is permanently protected against erosion or a permanent detention basin is constructed.

Sediment basins are suitable for nearly all types of construction projects. Whenever possible, construct the sediment basins before clearing and grading work begins. Basins should be located at the stormwater outlet from the site but not in any natural or undisturbed stream. A typical application would include temporary dikes, pipes, and/or channels to divert runoff to the basin inlet.

Many development projects in California will be required by local ordinances to provide a stormwater detention basin for post-construction flood control, desilting, or stormwater pollution control. A temporary sediment basin may be constructed by rough grading the post-construction control basins early in the project.

Sediment basins trap 70-80 % of the sediment that flows into them if designed according to this handbook. Therefore, they should be used in conjunction with erosion control practices such as

temporary seeding, mulching, diversion dikes, etc., to reduce the amount of sediment flowing into the basin.

Planning

To improve the effectiveness of the basin, it should be located to intercept runoff from the largest possible amount of disturbed area. The best locations are generally low areas. Drainage into the basin can be improved by the use of earth dikes and drainage swales (see BMP EC-9). The basin must not be located in a stream but it should be located to trap sediment-laden runoff before it enters the stream. The basin should not be located where its failure would result in the loss of life or interruption of the use or service of public utilities or roads.

- Construct before clearing and grading work begins when feasible.
- Do not locate in a stream.
- Basin sites should be located where failure of the structure will not cause loss of life, damage to homes or buildings, or interruption of use or service of public roads or utilities.
- Large basins are subject to state and local dam safety requirements.
- Limit the contributing area to the sediment basin to only the runoff from the disturbed soil areas. Use temporary concentrated flow conveyance controls to divert runoff from undisturbed areas away from the sediment basin.
- The basin should be located: (1) by excavating a suitable area or where a low embankment can be constructed across a swale, (2) where post-construction (permanent) detention basins will be constructed, and (3) where the basins can be maintained on a year-round basis to provide access for maintenance, including sediment removal and sediment stockpiling in a protected area, and to maintain the basin to provide the required capacity.

Design

Sediment basins must be designed in accordance with Section A of the State of California NPDES General Permit for Stormwater Discharges Associated with Construction Activities (General Permit) where sediment basins are the only control measure proposed for the site. If there is insufficient area to construct a sediment basin in accordance with the General Permit requirements, then the alternate design standards specified herein may be used.

Sediment basins designed per the General Permit shall be designed as follows:

Option 1:

Pursuant to local ordinance for sediment basin design and maintenance, provided that the design efficiency is as protective or more protective of water quality than Option 3.

OR

Option 2:

Sediment basin(s), as measured from the bottom of the basin to the principal outlet, shall have at least a capacity equivalent to 3,600 cubic feet (133 yd^3) of storage per acre draining into the sediment basin. The length of the basin shall be more than twice the width of the basin. The

length is determined by measuring the distance between the inlet and the outlet; and the depth must not be less than 3 ft nor greater than 5 ft for safety reasons and for maximum efficiency.

OR

Option 3:
Sediment basin(s) shall be designed using the standard equation:

$$As=1.2Q/Vs \quad \text{(Eq. 1)}$$

Where:

As = Minimum surface area for trapping soil particles of a certain size

Vs = Settling velocity of the design particle size chosen

Q = C I A

Where

Q = Discharge rate measured in cubic feet per second

C = Runoff coefficient

I = Precipitation intensity for the 10-year, 6-hour rain event

A = Area draining into the sediment basin in acres

The design particle size shall be the smallest soil grain size determined by wet sieve analysis, or the fine silt sized (0.01 mm [or 0.0004 in.]) particle, and the *Vs* used shall be 100 percent of the calculated settling velocity.

The length is determined by measuring the distance between the inlet and the outlet; the length shall be more than twice the dimension as the width; the depth shall not be less than 3 ft nor greater than 5 ft for safety reasons and for maximum efficiency (2 ft of sediment storage, 2 ft of capacity). The basin(s) shall be located on the site where it can be maintained on a year-round basis and shall be maintained on a schedule to retain the 2 ft of capacity.

OR

Option 4:
The use of an equivalent surface area design or equation, provided that the design efficiency is as protective or more protective of water quality than Option 3.

Other design considerations are:

- The volume of the settling zone should be sized to capture runoff from a 2-year storm or other appropriate design storms specified by the local agency. A detention time of 24 to 40 hours should allow 70 to 80 % of sediment to settle.
- The basin volume consists of two zones:
  - A sediment storage zone at least 1 ft deep.
  - A settling zone at least 2 ft deep.
- The length to settling depth ratio (L/SD) should be less than 200.
- Sediment basins are best used in conjunction with erosion controls. Sediment basins that will be used as the only means of treatment, without upstream erosion and sediment controls, must be designed according to the four options required by the General Permit (see Options 1-4 above). Sediment basins that are used in conjunction with upstream erosion and sediment controls should be designed to have a capacity equivalent to 67 yd^3 of sediment storage per acre of contributory area.
- The length of the basin should be more than twice the width of the basin; the length should be determined by measuring the distance between the inlet and the outlet.
- The depth must be no less than 3 ft.
- Basins with an impounding levee greater than 4.5 ft tall, measured from the lowest point to the impounding area to the highest point of the levee, and basins capable of impounding more than 35,000 ft^3, should be designed by a Registered Civil Engineer. The design should include maintenance requirements, including sediment and vegetation removal, to ensure continuous function of the basin outlet and bypass structures.
- Basins should be designed to drain within 72 hours following storm events. If a basin fails to drain within 72 hours, it must be pumped dry.
- Sediment basins, regardless of size and storage volume, should include features to accommodate overflow or bypass flows that exceed the design storm event.
  - Include an emergency spillway to accommodate flows not carried by the principal spillway. The spillway should consist of an open channel (earthen or vegetated) over undisturbed material (not fill) or constructed of a non-erodible riprap.
  - The spillway control section, which is a level portion of the spillway channel at the highest elevation in the channel, should be a minimum of 20 ft in length.
- Rock or vegetation should be used to protect the basin inlet and slopes against erosion.
- A forebay, constructed upstream of the basin may be provided to remove debris and larger particles.

- The outflow from the sediment basin should be provided with velocity dissipation devices (see BMP EC-10) to prevent erosion and scouring of the embankment and channel.
- Basin inlets should be located to maximize travel distance to the basin outlet.
- The principal outlet should consist of a corrugated metal, high density polyethylene (HDPE), or reinforced concrete riser pipe with dewatering holes and an anti-vortex device and trash rack attached to the top of the riser, to prevent floating debris from flowing out of the basin or obstructing the system. This principal structure should be designed to accommodate the inflow design storm.
- A rock pile or rock-filled gabions can serve as alternatives to the debris screen; although the designer should be aware of the potential for extra maintenance involved should the pore spaces in the rock pile clog.
- The outlet structure should be placed on a firm, smooth foundation with the base securely anchored with concrete or other means to prevent floatation.
- Attach riser pipe (watertight connection) to a horizontal pipe (barrel). Provide anti-seep collars on the barrel.
- Cleanout level should be clearly marked on the riser pipe.
- Proper hydraulic design of the outlet is critical to achieving the desired performance of the basin. The outlet should be designed to drain the basin within 24 to 72 hours (also referred to as "drawdown time"). The 24-hour limit is specified to provide adequate settling time; the 72-hour limit is specified to mitigate vector control concerns.
- The two most common outlet problems that occur are: (1) the capacity of the outlet is too great resulting in only partial filling of the basin and drawdown time less than designed for; and (2) the outlet clogs because it is not adequately protected against trash and debris. To avoid these problems, the following outlet types are recommended for use: (1) a single orifice outlet with or without the protection of a riser pipe, and (2) perforated riser. Design guidance for single orifice and perforated riser outlets follow:
  - *Flow Control Using a Single Orifice At The Bottom Of The Basin (Figure 1)*: The outlet control orifice should be sized using the following equation:

$$a = \frac{2A(H-Ho)^{0.5}}{3600CT(2g)^{0.5}} = \frac{(7x10^{-5})A(H-Ho)^{0.5}}{CT} \qquad \text{(Eq. 2)}$$

where:

a = area of orifice (ft^2)

A = surface area of the basin at mid elevation (ft^2)

C = orifice coefficient

T = drawdown time of full basin (hrs)

g = gravity (32.2 ft/s²)

H = elevation when the basin is full (ft)

Ho = final elevation when basin is empty (ft)

With a drawdown time of 40 hours, the equation becomes:

$$a = \frac{(1.75x10^{-6})A(H - Ho)^{0.5}}{C} \qquad \text{(Eq. 3)}$$

- *Flow Control Using Multiple Orifices (see Figure2):*

$$a_t = \frac{2A(h_{max})}{3600CT(2g[h_{max} - h_{centroid\ of\ orifices}])^{0.5}} \qquad \text{(Eq. 4)}$$

With terms as described above except:

a_t = total area of orifices

h_{max} = maximum height from lowest orifice to the maximum water surface (ft)

$h_{centroid\ of\ orifices}$ = height from the lowest orifice to the centroid of the orifice configuration (ft)

Allocate the orifices evenly on two rows; separate the holes by 3x hole diameter vertically, and by 120 degrees horizontally (refer to Figure 2).

Because basins are not maintained for infiltration, water loss by infiltration should be disregarded when designing the hydraulic capacity of the outlet structure.

Care must be taken in the selection of "C"; 0.60 is most often recommended and used. However, based on actual tests, GKY (1989), "Outlet Hydraulics of Extended Detention Facilities for Northern Virginia Planning District Commission", recommends the following:

C = 0.66 for thin materials; where the thickness is equal to or less than the orifice diameter, or

C = 0.80 when the material is thicker than the orifice diameter

Installation

- Securely anchor and install an anti-seep collar on the outlet pipe/riser and provide an emergency spillway for passing major floods (see local flood control agency).
- Areas under embankments must be cleared and stripped of vegetation.
- Chain link fencing should be provided around each sediment basin to prevent unauthorized entry to the basin or if safety is a concern.

Costs

Average annual costs for installation and maintenance (2 year useful life) are:

- Basin less than 50,000 ft³: Range, $0.24 - $1.58/ft³. Average, $0.73 per ft³. $400 - $2,400, $1,200 average per drainage acre.
- Basin size greater than 50,000 ft³: Range, $0.12 – $0.48/ft³. Average, $0.36 per ft³. $200 - $800, $600 average per drainage acre.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Examine basin banks for seepage and structural soundness.
- Check inlet and outlet structures and spillway for any damage or obstructions. Repair damage and remove obstructions as needed.
- Check inlet and outlet area for erosion and stabilize if required.
- Check fencing for damage and repair as needed.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when sediment accumulation reaches one-half the designated sediment storage volume. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed of at appropriate locations.
- Remove standing water from basin within 72 hours after accumulation.
- BMPs that require dewatering shall be continuously attended while dewatering takes place. Dewatering BMPs shall be implemented at all times during dewatering activities.
- To minimize vector production:
  - Remove accumulation of live and dead floating vegetation in basins during every inspection.
  - Remove excessive emergent and perimeter vegetation as needed or as advised by local or state vector control agencies.

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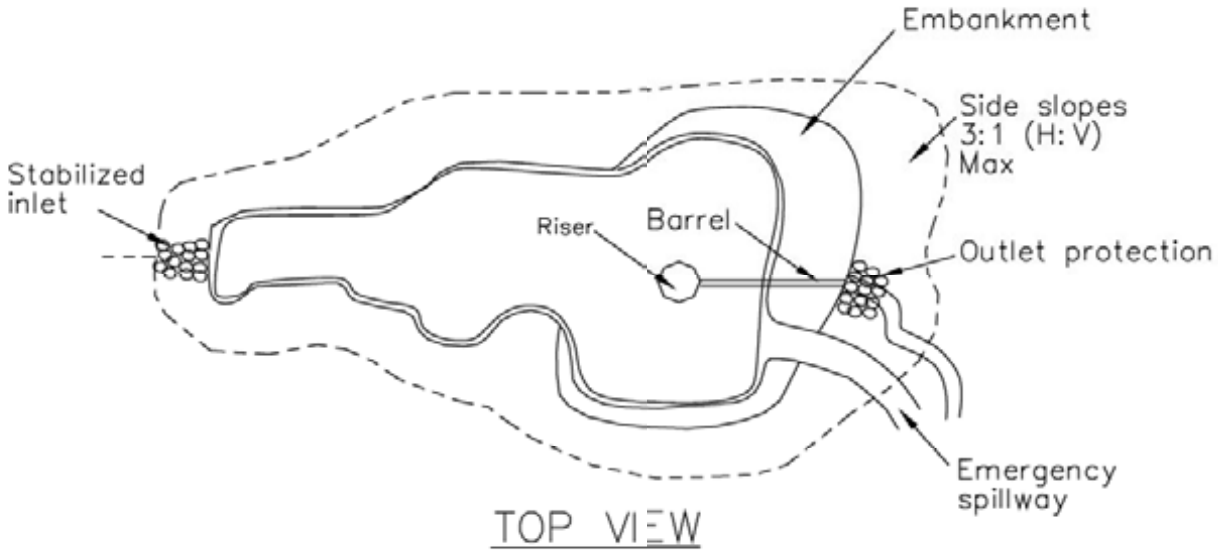
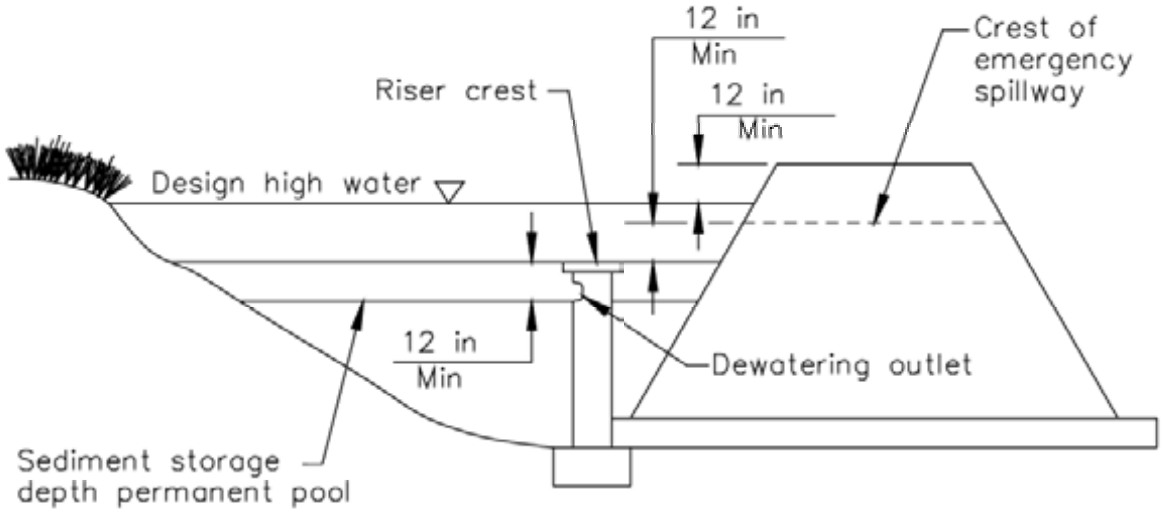


TOP VIEW



SIDE VIEW

NOTE:
This outlet provides no drainage for permanent pool.

FIGURE 1: TYPICAL TEMPORARY SEDIMENT BASIN
SINGLE ORIFICE DESIGN
NOT TO SCALE

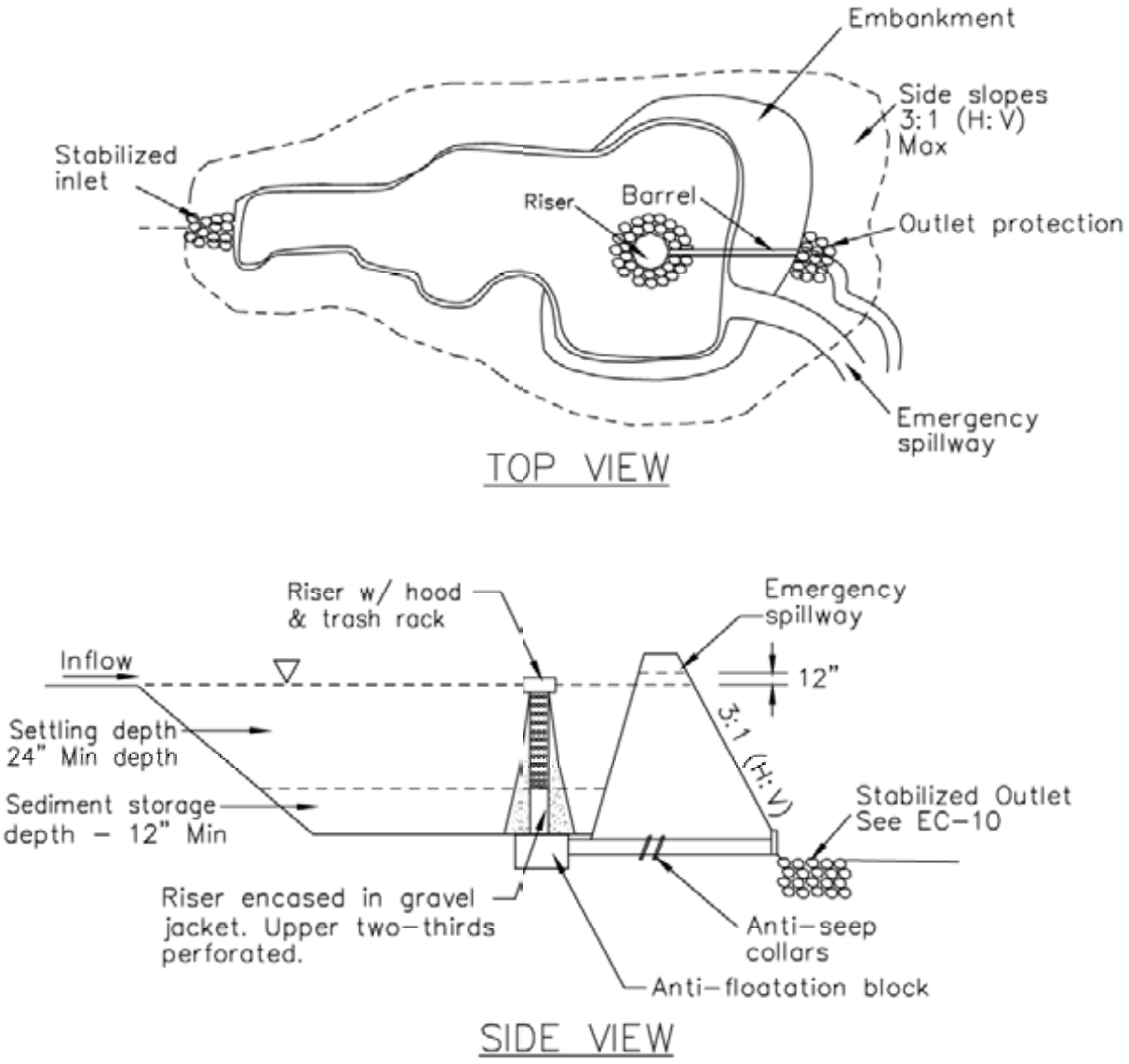


FIGURE 2: TYPICAL TEMPORARY SEDIMENT BASIN

MULTIPLE ORIFICE DESIGN

NOT TO SCALE

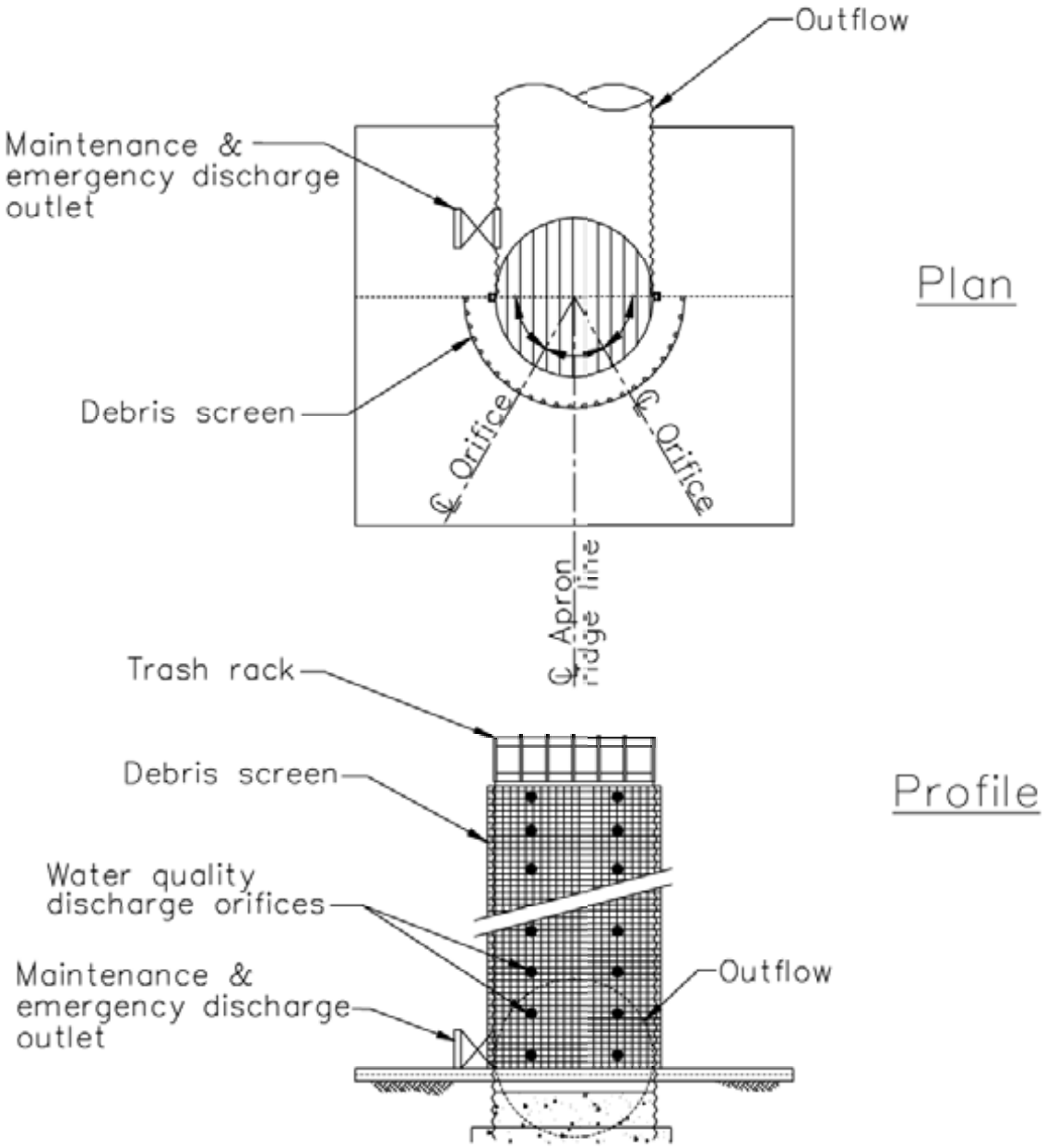
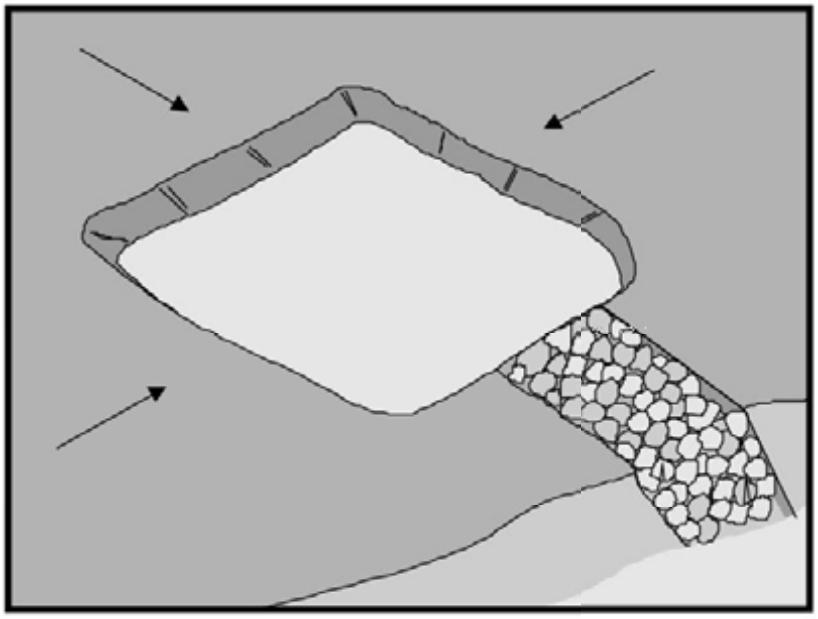


FIGURE 3: MULTIPLE ORIFICE OUTLET RISER

NOT TO SCALE

Sediment Trap SE-3



Objectives

EC	Erosion Control	
SE	Sediment Control	☑
TR	Tracking Control	
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Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-2 Sediment Basin (for larger areas)

Description and Purpose

A sediment trap is a containment area where sediment-laden runoff is temporarily detained under quiescent conditions, allowing sediment to settle out or before the runoff is discharged. Sediment traps are formed by excavating or constructing an earthen embankment across a waterway or low drainage area.

Suitable Applications

Sediment traps should be considered for use:

- At the perimeter of the site at locations where sediment-laden runoff is discharged offsite.
- At multiple locations within the project site where sediment control is needed.
- Around or upslope from storm drain inlet protection measures.
- Sediment traps may be used on construction projects where the drainage area is less than 5 acres. Traps would be placed where sediment-laden stormwater may enter a storm drain or watercourse. SE-2, Sediment Basins, must be used for drainage areas greater than 5 acres.
- As a supplemental control, sediment traps provide additional protection for a water body or for reducing sediment before it enters a drainage system.

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Limitations

- Requires large surface areas to permit infiltration and settling of sediment.
- Not appropriate for drainage areas greater than 5 acres.
- Only removes large and medium sized particles and requires upstream erosion control.
- Attractive and dangerous to children, requiring protective fencing.
- Conducive to vector production.
- Should not be located in live streams.

Implementation

Design

A sediment trap is a small temporary ponding area, usually with a gravel outlet, formed by excavation or by construction of an earthen embankment. Its purpose is to collect and store sediment from sites cleared or graded during construction. It is intended for use on small drainage areas with no unusual drainage features and projected for a quick build-out time. It should help in removing coarse sediment from runoff. The trap is a temporary measure with a design life of approximately six months to one year and is to be maintained until the site area is permanently protected against erosion by vegetation and/or structures.

Sediment traps should be used only for small drainage areas. If the contributing drainage area is greater than 5 acres, refer to SE-2, Sediment Basins, or subdivide the catchment area into smaller drainage basins.

Sediment usually must be removed from the trap after each rainfall event. The SWPPP should detail how this sediment is to be disposed of, such as in fill areas onsite, or removal to an approved offsite dump. Sediment traps used as perimeter controls should be installed before any land disturbance takes place in the drainage area.

Sediment traps are usually small enough that a failure of the structure would not result in a loss of life, damage to home or buildings, or interruption in the use of public roads or utilities. However, sediment traps are attractive to children and can be dangerous. The following recommendations should be implemented to reduce risks:

- Install continuous fencing around the sediment trap or pond. Consult local ordinances regarding requirements for maintaining health and safety.
- Restrict basin side slopes to 3:1 or flatter.

Sediment trap size depends on the type of soil, size of the drainage area, and desired sediment removal efficiency (see SE-2, Sediment Basin). As a rule of thumb, the larger the basin volume the greater the sediment removal efficiency. Sizing criteria are typically established under the local grading ordinance or equivalent. The runoff volume from a 2-year storm is a common design criteria for a sediment trap. The sizing criteria below assume that this runoff volume is 0.042 acre-ft/acre (0.5 in. of runoff). While the climatic, topographic, and soil type extremes make it difficult to establish a statewide standard, the following criteria should trap moderate to high amounts of sediment in most areas of California:

- Locate sediment traps as near as practical to areas producing the sediment.
- Trap should be situated according to the following criteria: (1) by excavating a suitable area or where a low embankment can be constructed across a swale, (2) where failure would not cause loss of life or property damage, and (3) to provide access for maintenance, including sediment removal and sediment stockpiling in a protected area.
- Trap should be sized to accommodate a settling zone and sediment storage zone with recommended minimum volumes of 67 yd^3/acre and 33 yd^3/acre of contributing drainage area, respectively, based on 0.5 in. of runoff volume over a 24-hour period. In many cases, the size of an individual trap is limited by available space. Multiple traps or additional volume may be required to accommodate specific rainfall, soil, and site conditions.
- Traps with an impounding levee greater than 4.5 ft tall, measured from the lowest point to the impounding area to the highest point of the levee, and traps capable of impounding more than 35,000 ft^3, should be designed by a Registered Civil Engineer. The design should include maintenance requirements, including sediment and vegetation removal, to ensure continuous function of the trap outlet and bypass structures.
- The outlet pipe or open spillway must be designed to convey anticipated peak flows.
- Use rock or vegetation to protect the trap outlets against erosion.
- Fencing should be provided to prevent unauthorized entry.

Installation

Sediment traps can be constructed by excavating a depression in the ground or creating an impoundment with a small embankment. Sediment traps should be installed outside the area being graded and should be built prior to the start of the grading activities or removal of vegetation. To minimize the area disturbed by them, sediment traps should be installed in natural depressions or in small swales or drainage ways. The following steps must be followed during installation:

- The area under the embankment must be cleared, grubbed, and stripped of any vegetation and root mat. The pool area should be cleared.
- The fill material for the embankment must be free of roots or other woody vegetation as well as oversized stones, rocks, organic material, or other objectionable material. The embankment may be compacted by traversing with equipment while it is being constructed.
- All cut-and-fill slopes should be 3:1 or flatter.
- When a riser is used, all pipe joints must be watertight.
- When a riser is used, at least the top two-thirds of the riser should be perforated with 0.5 in. diameter holes spaced 8 in. vertically and 10 to 12 in. horizontally. See SE-2, Sediment Basin.
- When an earth or stone outlet is used, the outlet crest elevation should be at least 1 ft below the top of the embankment.

- When crushed stone outlet is used, the crushed stone used in the outlet should meet AASHTO M43, size No. 2 or 24, or its equivalent such as MSHA No. 2. Gravel meeting the above gradation may be used if crushed stone is not available.

Costs

Average annual cost per installation and maintenance (18 month useful life) is $0.73 per ft^3 ($1,300 per drainage acre). Maintenance costs are approximately 20% of installation costs.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Inspect outlet area for erosion and stabilize if required.
- Inspect trap banks for seepage and structural soundness, repair as needed.
- Inspect outlet structure and spillway for any damage or obstructions. Repair damage and remove obstructions as needed.
- Inspect fencing for damage and repair as needed.
- Inspect the sediment trap for area of standing water during every visit. Corrective measures should be taken if the BMP does not dewater completely in 72 hours or less to prevent vector production.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the trap capacity. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed of at an appropriate location.
- Remove vegetation from the sediment trap when first detected to prevent pools of standing water and subsequent vector production.
- BMPs that require dewatering shall be continuously attended while dewatering takes place. Dewatering BMPs shall be implemented at all times during dewatering activities.

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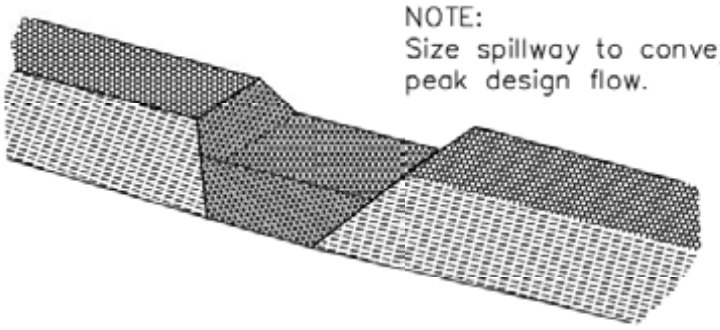
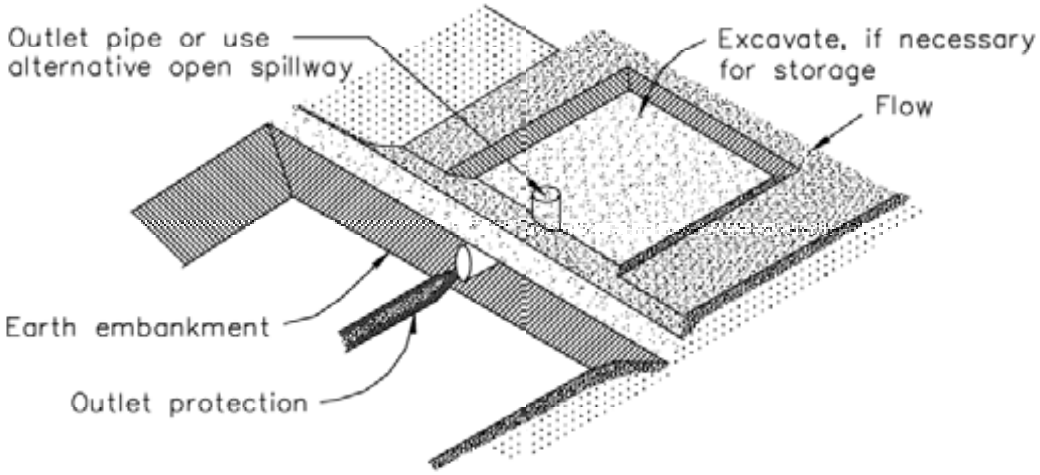
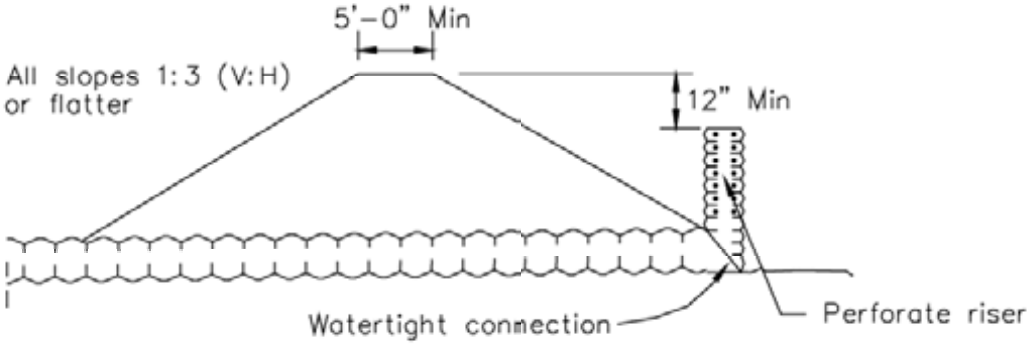


TYPICAL OPEN SPILLWAY

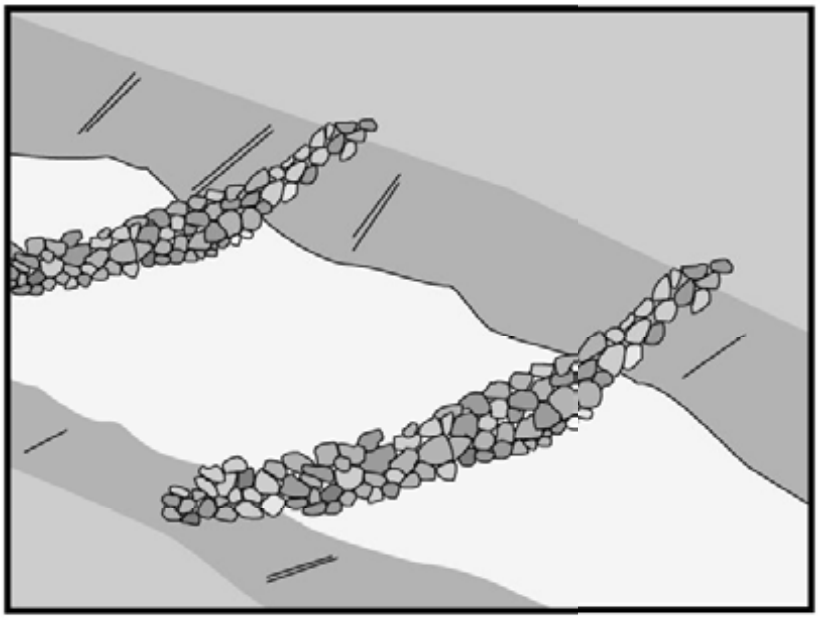




EMBANKMENT SECTION THRU RISER

TYPICAL SEDIMENT TRAP
NOT TO SCALE

Check Dams SE-4



Objectives

EC	Erosion Control	☒
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-8 Sandbag Barrier

Description and Purpose

A check dam is a small barrier constructed of rock, gravel bags, sandbags, fiber rolls, or reusable products, placed across a constructed swale or drainage ditch. Check dams reduce the effective slope of the channel, thereby reducing the velocity of flowing water, allowing sediment to settle and reducing erosion.

Suitable Applications

Check dams may be appropriate in the following situations:

- To promote sedimentation behind the dam.
- To prevent erosion by reducing the velocity of channel flow in small intermittent channels and temporary swales.
- In small open channels that drain 10 acres or less.
- In steep channels where stormwater runoff velocities exceed 5 ft/s.
- During the establishment of grass linings in drainage ditches or channels.
- In temporary ditches where the short length of service does not warrant establishment of erosion-resistant linings.

Limitations

- Not to be used in live streams or in channels with extended base flows.

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- Not appropriate in channels that drain areas greater than 10 acres.
- Not appropriate in channels that are already grass-lined unless erosion is expected, as installation may damage vegetation.
- Require extensive maintenance following high velocity flows.
- Promotes sediment trapping which can be re-suspended during subsequent storms or removal of the check dam.

Implementation

General

Check dams reduce the effective slope and create small pools in swales and ditches that drain 10 acres or less. Reduced slopes reduce the velocity of stormwater flows, thus reducing erosion of the swale or ditch and promoting sedimentation. Use of check dams for sedimentation will likely result in little net removal of sediment because of the small detention time and probable scour during longer storms. Using a series of check dams will generally increase their effectiveness. A sediment trap (SE-3) may be placed immediately upstream of the check dam to increase sediment removal efficiency.

Design and Layout

Check dams work by decreasing the effective slope in ditches and swales. An important consequence of the reduced slope is a reduction in capacity of the ditch or swale. This reduction in capacity must be considered when using this BMP, as reduced capacity can result in overtopping of the ditch or swale and resultant consequences. In some cases, such as a "permanent" ditch or swale being constructed early and used as a "temporary" conveyance for construction flows, the ditch or swale may have sufficient capacity such that the temporary reduction in capacity due to check dams is acceptable. When check dams reduce capacities beyond acceptable limits, there are several options:

- Don't use check dams. Consider alternative BMPs.
- Increase the size of the ditch or swale to restore capacity.

Maximum slope and velocity reduction is achieved when the toe of the upstream dam is at the same elevation as the top of the downstream dam. The center section of the dam should be lower than the edge sections so that the check dam will direct flows to the center of the ditch or swale.

Check dams are usually constructed of rock, gravel bags, sandbags, and fiber rolls. A number of products manufactured specifically for use as check dams are also being used, and some of these products can be removed and reused. Check dams can also be constructed of logs or lumber, and have the advantage of a longer lifespan when compared to gravel bags, sandbags, and fiber rolls. Straw bales can also be used for check dams and can work if correctly installed; but in practice, straw bale check dams have a high failure rate. Check dams should not be constructed from straw bales or silt fences, since concentrated flows quickly wash out these materials.

Rock check dams are usually constructed of 8 to 12 in. rock. The rock is placed either by hand or mechanically, but never just dumped into the channel. The dam must completely span the ditch

or swale to prevent washout. The rock used must be large enough to stay in place given the expected design flow through the channel.

Log check dams are usually constructed of 4 to 6 in. diameter logs. The logs should be embedded into the soil at least 18 in. Logs can be bolted or wired to vertical support logs that have been driven or buried into the soil.

Gravel bag and sandbag check dams are constructed by stacking bags across the ditch or swale, shaped as shown in the drawings at the end of this fact sheet.

Manufactured products should be installed in accordance with the manufacturer's instructions.

If grass is planted to stabilize the ditch or swale, the check dam should be removed when the grass has matured (unless the slope of the swales is greater than 4%).

The following guidance should be followed for the design and layout of check dams:

- Install the first check dam approximately 16 ft from the outfall device and at regular intervals based on slope gradient and soil type.
- Check dams should be placed at a distance and height to allow small pools to form between each check dam.
- Backwater from a downstream check dam should reach the toes of the upstream check dam.
- A sediment trap provided immediately upstream of the check dam will help capture sediment. Due to the potential for this sediment to be resuspended in subsequent storms, the sediment trap must be cleaned following each storm event.
- High flows (typically a 2-year storm or larger) should safely flow over the check dam without an increase in upstream flooding or damage to the check dam.
- Where grass is used to line ditches, check dams should be removed when grass has matured sufficiently to protect the ditch or swale.
- Gravel bags may be used as check dams with the following specifications:

Materials

Gravel bags used for check dams should conform to the requirements of SE-6, Gravel Bag Berms. Sandbags used for check dams should conform to SE-8, Sandbag Barrier. Fiber rolls used for check dams should conform to SE-5, Fiber Rolls. Straw bales used for check dams should conform to SE-9, Straw Bale Barrier.

Installation

- Rock should be placed individually by hand or by mechanical methods (no dumping of rock) to achieve complete ditch or swale coverage.
- Tightly abut bags and stack according to detail shown in the figure at the end of this section. Gravel bags and sandbags should not be stacked any higher than 3 ft.
- Fiber rolls and straw bales must be trenched in and firmly staked in place.

Costs

Cost consists of only installation costs if materials are readily available. If material must be imported, costs may increase. For material costs, see SE-5, SE-6, SE-8 and SE-9.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Replace missing rock, bags, bales, etc. Replace bags or bales that have degraded or have become damaged.
- If the check dam is used as a sediment capture device, sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed at an appropriate location.
- If the check dam is used as a grade control structure, sediment removal is not required as long as the system continues to control the grade.
- Remove accumulated sediment prior to permanent seeding or soil stabilization.
- Remove check dam and accumulated sediment when check dams are no longer needed.

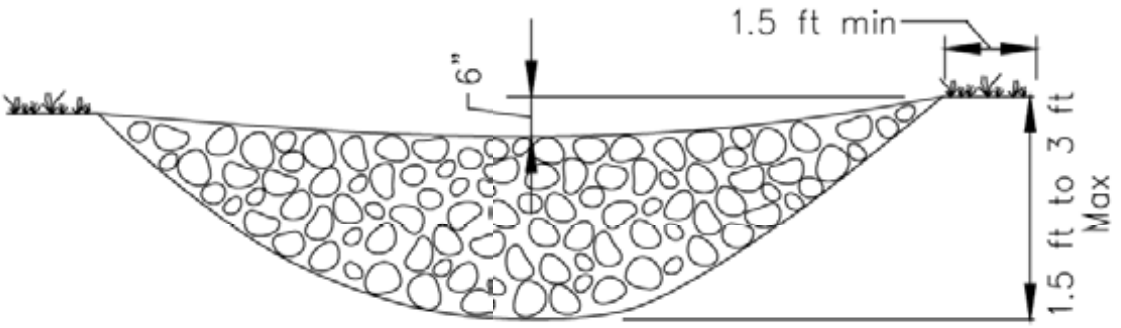
References

Draft – Sedimentation and Erosion Control, and Inventory of Current Practices, USEPA, April 1990.

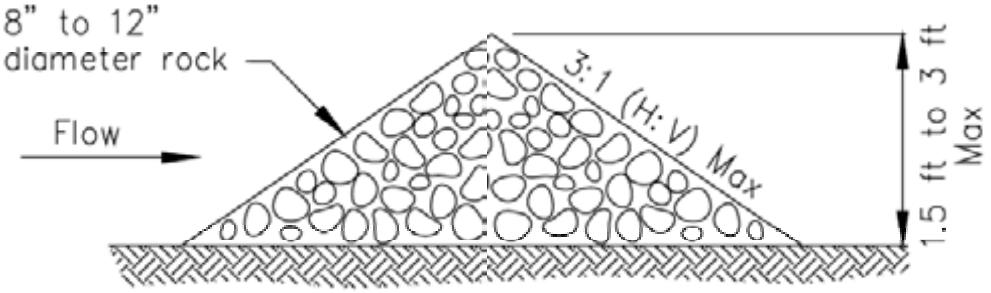
Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stormwater Management of the Puget Sound Basin, Technical Manual, Publication #91-75, Washington State Department of Ecology, February 1992.

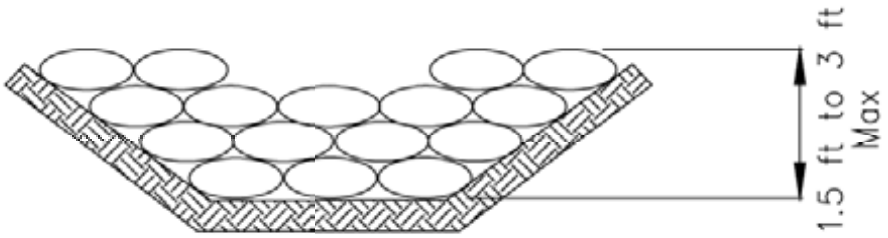


ELEVATION



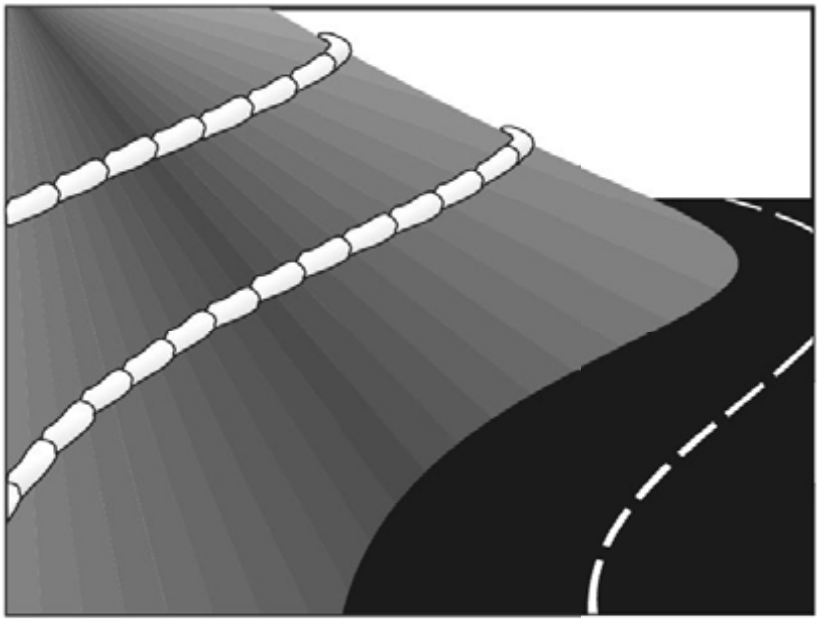
TYPICAL ROCK CHECK DAM SECTION

ROCK CHECK DAM
NOT TO SCALE



GRAVEL BAG CHECK DAM ELEVATION
NOT TO SCALE

Gravel Bag Berm SE-6



Objectives

EC	Erosion Control	☒
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence

SE-5 Fiber Roll

SE-8 Sandbag Barrier

SE-9 Straw Bale Barrier

Description and Purpose

A gravel bag berm is a series of gravel-filled bags placed on a level contour to intercept sheet flows. Gravel bags pond sheet flow runoff, allowing sediment to settle out, and release runoff slowly as sheet flows, preventing erosion.

Suitable Applications

Gravel bag berms may be suitable:

- As a linear sediment control measure:
  - Below the toe of slopes and erodible slopes
  - As sediment traps at culvert/pipe outlets
  - Below other small cleared areas
  - Along the perimeter of a site
  - Down slope of exposed soil areas
  - Around temporary stockpiles and spoil areas
  - Parallel to a roadway to keep sediment off paved areas
  - Along streams and channels
- As linear erosion control measure:

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- Along the face and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff as sheet flow
- At the top of slopes to divert runoff away from disturbed slopes
- As check dams across mildly sloped construction roads

Limitations

- Gravel berms may be difficult to remove.
- Removal problems limit their usefulness in landscaped areas.
- Gravel bag berm may not be appropriate for drainage areas greater than 5 acres.
- Runoff will pond upstream of the filter, possibly causing flooding if sufficient space does not exist.
- Degraded gravel bags may rupture when removed, spilling contents.
- Installation can be labor intensive.
- Berms may have limited durability for long-term projects.
- When used to detain concentrated flows, maintenance requirements increase.

Implementation

General

A gravel bag berm consists of a row of open graded gravel–filled bags placed on a level contour. When appropriately placed, a gravel bag berm intercepts and slows sheet flow runoff, causing temporary ponding. The temporary ponding provides quiescent conditions allowing sediment to settle. The open graded gravel in the bags is porous, which allows the ponded runoff to flow slowly through the bags, releasing the runoff as sheet flows. Gravel bag berms also interrupt the slope length and thereby reduce erosion by reducing the tendency of sheet flows to concentrate into rivulets, which erode rills, and ultimately gullies, into disturbed, sloped soils. Gravel bag berms are similar to sand bag barriers, but are more porous.

Design and Layout

- Locate gravel bag berms on level contours.
  - Slopes between 20:1 and 2:1 (H:V): Gravel bags should be placed at a maximum interval of 50 ft (a closer spacing is more effective), with the first row near the slope toe.
  - Slopes 2:1 (H:V) or steeper: Gravel bags should be placed at a maximum interval of 25 ft (a closer spacing is more effective), with the first row placed the slope toe.
- Turn the ends of the gravel bag barriers up slope to prevent runoff from going around the berm.
- Allow sufficient space up slope from the gravel bag berm to allow ponding, and to provide room for sediment storage.

- For installation near the toe of the slope, consider moving the gravel bag barriers away from the slope toe to facilitate cleaning. To prevent flows behind the barrier, bags can be placed perpendicular to a berm to serve as cross barriers.
- Drainage area should not exceed 5 acres.
- In Non-Traffic Areas:
  - Height = 18 in. maximum
  - Top width = 24 in. minimum for three or more layer construction
  - Top width = 12 in. minimum for one or two layer construction
  - Side slopes = 2:1 or flatter
- In Construction Traffic Areas:
  - Height = 12 in. maximum
  - Top width = 24 in. minimum for three or more layer construction.
  - Top width = 12 in. minimum for one or two layer construction.
  - Side slopes = 2:1 or flatter.
- Butt ends of bags tightly
- On multiple row, or multiple layer construction, overlapp butt joints of adjacent row and row beneath.
- Use a pyramid approach when stacking bags.

Materials

- ***Bag Material:*** Bags should be woven polypropylene, polyethylene or polyamide fabric or burlap, minimum unit weight of 4 ounces/yd^2, Mullen burst strength exceeding 300 lb/in^2 in conformance with the requirements in ASTM designation D3786, and ultraviolet stability exceeding 70% in conformance with the requirements in ASTM designation D4355.
- ***Bag Size:*** Each gravel-filled bag should have a length of 18 in., width of 12 in., thickness of 3 in., and mass of approximately 33 lbs. Bag dimensions are nominal, and may vary based on locally available materials.
- ***Fill Material:*** Fill material should be 0.5 to 1 in. Class 2 aggregate base, clean and free from clay, organic matter, and other deleterious material, or other suitable open graded, non-cohesive, porous gravel.

Costs

Gravel filter: Expensive, since off-site materials, hand construction, and demolition/removal are usually required. Material costs for gravel bags are average of $2.50 per empty gravel bag. Gravel costs range from $20-$35 per yd^3.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Gravel bags exposed to sunlight will need to be replaced every two to three months due to degrading of the bags.
- Reshape or replace gravel bags as needed.
- Repair washouts or other damage as needed.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed at an appropriate location.
- Remove gravel bag berms when no longer needed. Remove sediment accumulation and clean, re-grade, and stabilize the area. Removed sediment should be incorporated in the project or disposed of.

References

Handbook of Steel Drainage and Highway Construction, American Iron and Steel Institute, 1983.

Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stormwater Pollution Plan Handbook, First Edition, State of California, Department of Transportation Division of New Technology, Materials and Research, October 1992.

Street Sweeping and Vacuuming SE-7

Objectives		
EC	Erosion Control	
SE	Sediment Control	☒
TR	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	☑
Organics	

Potential Alternatives

None

Description and Purpose

Street sweeping and vacuuming includes use of self-propelled and walk-behind equipment to remove sediment from streets and roadways, and to clean paved surfaces in preparation for final paving. Sweeping and vacuuming prevents sediment from the project site from entering storm drains or receiving waters.

Suitable Applications

Sweeping and vacuuming are suitable anywhere sediment is tracked from the project site onto public or private paved streets and roads, typically at points of egress. Sweeping and vacuuming are also applicable during preparation of paved surfaces for final paving.

Limitations

Sweeping and vacuuming may not be effective when sediment is wet or when tracked soil is caked (caked soil may need to be scraped loose).

Implementation

- Controlling the number of points where vehicles can leave the site will allow sweeping and vacuuming efforts to be focused, and perhaps save money.
- Inspect potential sediment tracking locations daily.
- Visible sediment tracking should be swept or vacuumed on a daily basis.

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- Do not use kick brooms or sweeper attachments. These tend to spread the dirt rather than remove it.
- If not mixed with debris or trash, consider incorporating the removed sediment back into the project

Costs

Rental rates for self-propelled sweepers vary depending on hopper size and duration of rental. Expect rental rates from $58/hour (3 yd³ hopper) to $88/hour (9 yd³ hopper), plus operator costs. Hourly production rates vary with the amount of area to be swept and amount of sediment. Match the hopper size to the area and expect sediment load to minimize time spent dumping.

Inspection and Maintenance

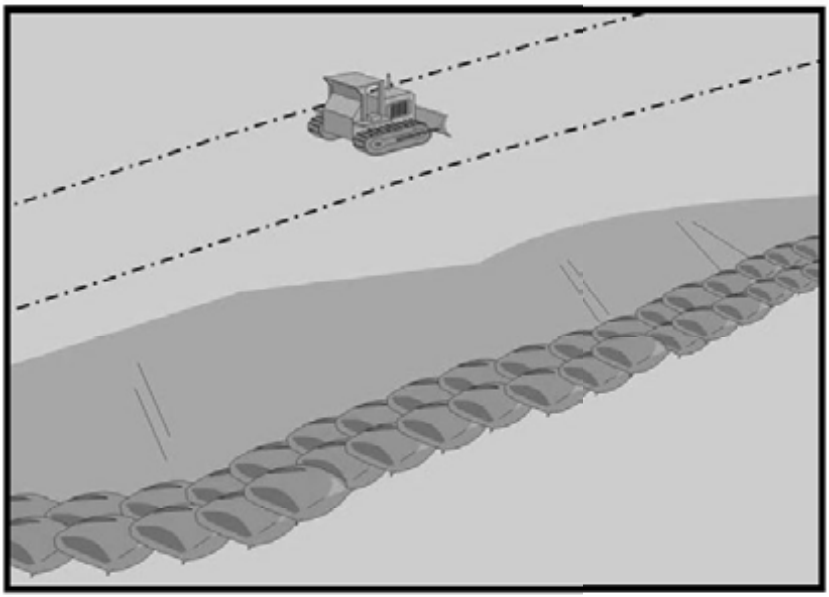
- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- When actively in use, points of ingress and egress must be inspected daily.
- When tracked or spilled sediment is observed outside the construction limits, it must be removed at least daily. More frequent removal, even continuous removal, may be required in some jurisdictions.
- Be careful not to sweep up any unknown substance or any object that may be potentially hazardous.
- Adjust brooms frequently; maximize efficiency of sweeping operations.
- After sweeping is finished, properly dispose of sweeper wastes at an approved dumpsite.

References

Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Labor Surcharge and Equipment Rental Rates, State of California Department of Transportation (Caltrans), April 1, 2002 – March 31, 2003.

Sandbag Barrier SE-8



Objectives

EC	Erosion Control	☒
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Description and Purpose

A sandbag barrier is a series of sand-filled bags placed on a level contour to intercept sheet flows. Sandbag barriers pond sheet flow runoff, allowing sediment to settle out.

Suitable Applications

Sandbag barriers may be suitable:

- As a linear sediment control measure:
  - Below the toe of slopes and erodible slopes
  - As sediment traps at culvert/pipe outlets
  - Below other small cleared areas
  - Along the perimeter of a site
  - Down slope of exposed soil areas
  - Around temporary stockpiles and spoil areas
  - Parallel to a roadway to keep sediment off paved areas
  - Along streams and channels
- As linear erosion control measure:
  - Along the face and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff as sheet flow

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-9 Straw Bale Barrier

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- At the top of slopes to divert runoff away from disturbed slopes
- As check dams across mildly sloped construction roads

Limitations

- It is necessary to limit the drainage area upstream of the barrier to 5 acres.
- Degraded sandbags may rupture when removed, spilling sand.
- Installation can be labor intensive.
- Barriers may have limited durability for long-term projects.
- When used to detain concentrated flows, maintenance requirements increase.
- Burlap should not be used for sandbags.

Implementation

General

A sandbag barrier consists of a row of sand-filled bags placed on a level contour. When appropriately placed, a sandbag barrier intercepts and slows sheet flow runoff, causing temporary ponding. The temporary ponding provides quiescent conditions allowing sediment to settle. While the sand-filled bags are porous, the fine sand tends to quickly plug with sediment, limiting the rate of flow through the barrier. If a porous barrier is desired, consider SE-1, Silt Fence, SE-5, Fiber Rolls, SE-6, Gravel Bag Berms, or SE-9, Straw Bale Barriers. Sandbag barriers also interrupt the slope length and thereby reduce erosion by reducing the tendency of sheet flows to concentrate into rivulets which erode rills, and ultimately gullies, into disturbed, sloped soils. Sandbag barriers are similar to ground bag berms, but less porous.

Design and Layout

- Locate sandbag barriers on a level contour.
  - Slopes between 20:1 and 2:1 (H:V): Sandbags should be placed at a maximum interval of 50 ft (a closer spacing is more effective), with the first row near the slope toe.
  - Slopes 2:1 (H:V) or steeper: Sandbags should be placed at a maximum interval of 25 ft (a closer spacing is more effective), with the first row placed near the slope toe.
- Turn the ends of the sandbag barrier up slope to prevent runoff from going around the barrier.
- Allow sufficient space up slope from the barrier to allow ponding, and to provide room for sediment storage.
- For installation near the toe of the slope, consider moving the barrier away from the slope toe to facilitate cleaning. To prevent flow behind the barrier, sandbags can be placed perpendicular to the barrier to serve as cross barriers.
- Drainage area should not exceed 5 acres.

- Stack sandbags at least three bags high.
- Butt ends of bags tightly.
- Overlapp butt joints of row beneath with each successive row.
- Use a pyramid approach when stacking bags.
- In non-traffic areas
  - Height = 18 in. maximum
  - Top width = 24 in. minimum for three or more layer construction
  - Side slope = 2:1 or flatter
- In construction traffic areas
  - Height = 12 in. maximum
  - Top width = 24 in. minimum for three or more layer construction.
  - Side slopes = 2:1 or flatter.

Materials

- ***Sandbag Material:*** Sandbag should be woven polypropylene, polyethylene or polyamide fabric, minimum unit weight of 4 ounces/yd², Mullen burst strength exceeding 300 lb/in² in conformance with the requirements in ASTM designation D3786, and ultraviolet stability exceeding 70% in conformance with the requirements in ASTM designation D4355. Use of burlap may not acceptable in some jurisdictions.
- ***Sandbag Size:*** Each sand-filled bag should have a length of 18 in., width of 12 in., thickness of 3 in., and mass of approximately 33 lbs. Bag dimensions are nominal, and may vary based on locally available materials.
- ***Fill Material:*** All sandbag fill material should be non-cohesive, Class 1 or Class 2 permeable material free from clay and deleterious material.

Costs

Sandbag barriers are more costly, but typically have a longer useful life than other barriers. Empty sandbags cost $0.25 - $0.75. Average cost of fill material is $8 per yd³. Pre-filled sandbags are more expensive at $1.50 - $2.00 per bag.

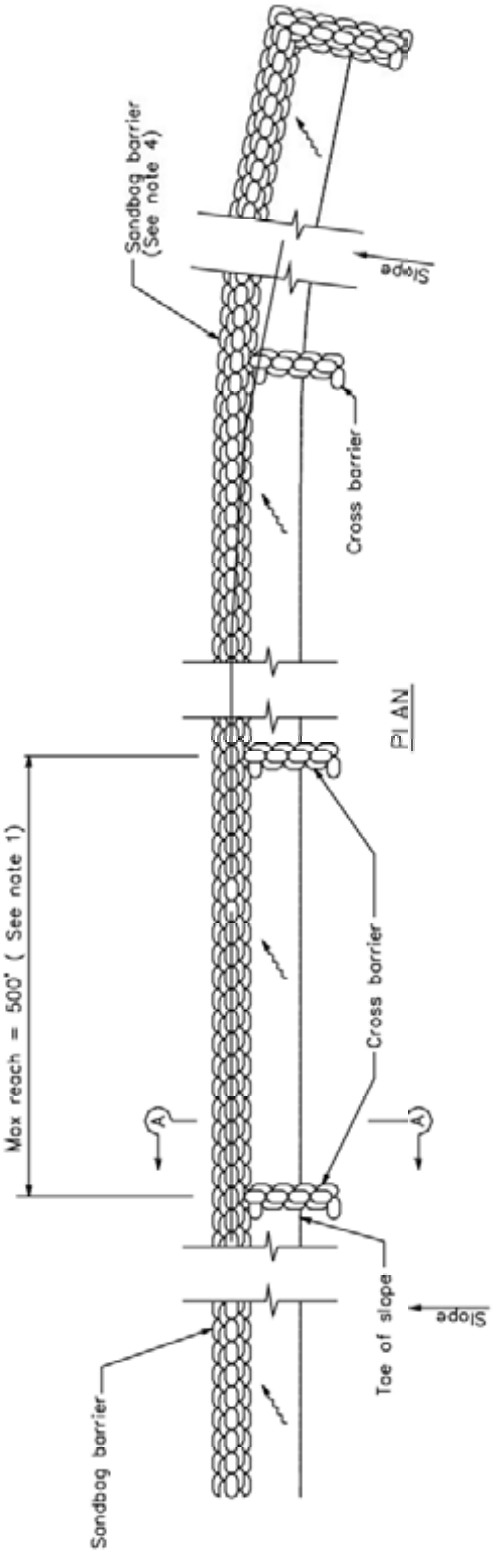
Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Sandbags exposed to sunlight will need to be replaced every two to three months due to degradation of the bags.
- Reshape or replace sandbags as needed.

- Repair washouts or other damage as needed.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed at an appropriate location.
- Remove sandbags when no longer needed. Remove sediment accumulation, and clean, re-grade, and stabilize the area.

References

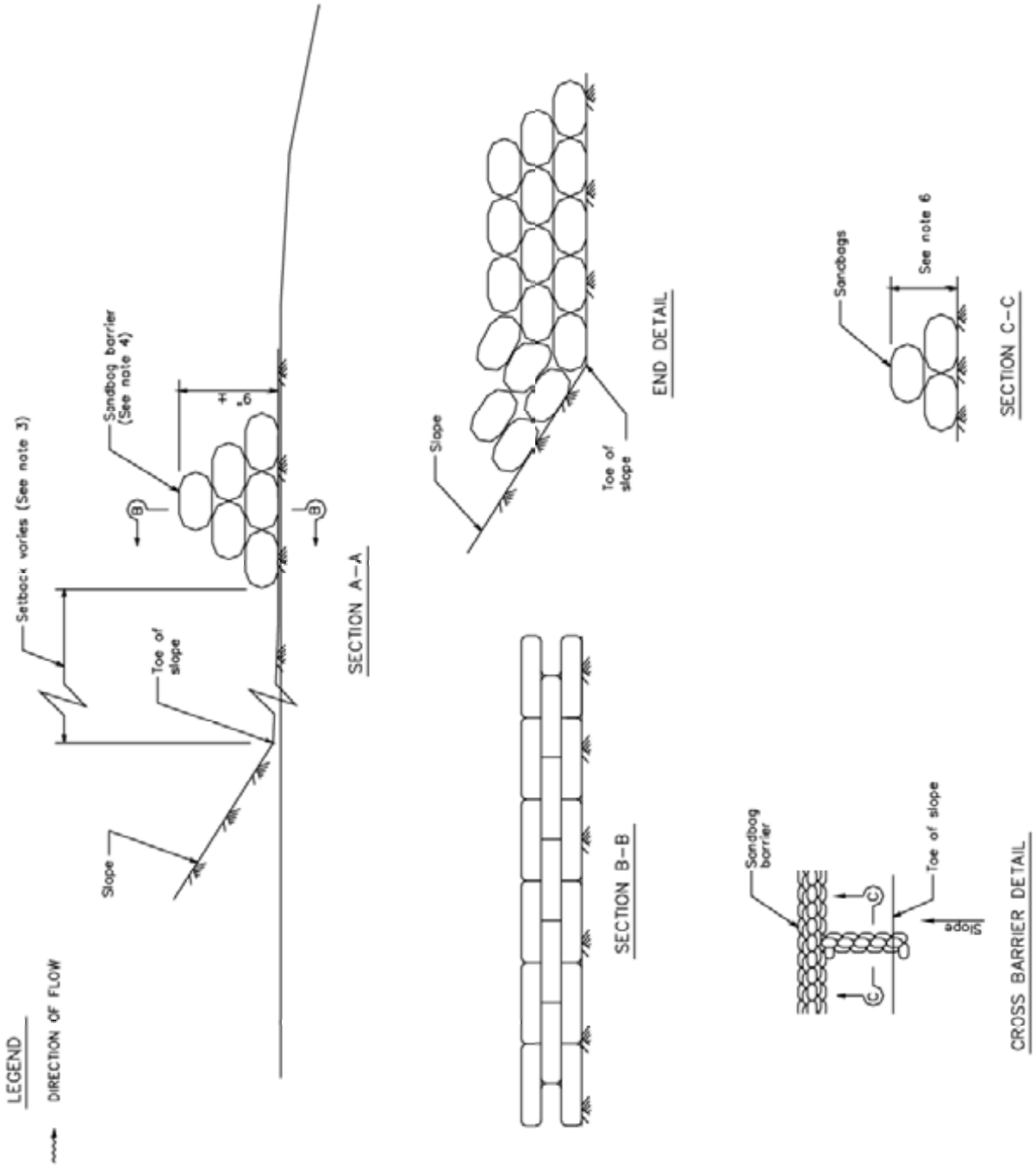
Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.



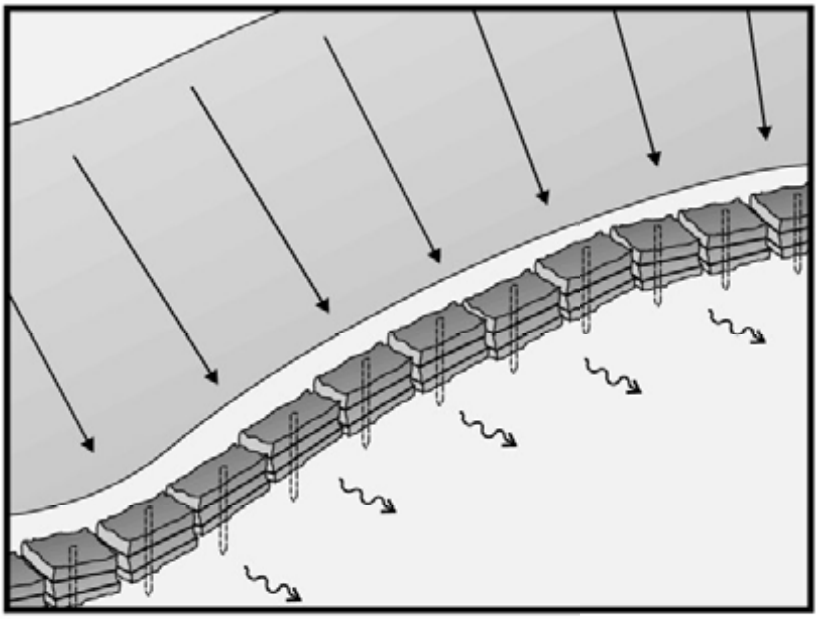
SANDBAG BARRIER

NOTES

1. Construct the length of each reach so that the change in base elevation along the reach does not exceed 1/2 the height of the linear barrier. In no case shall the reach length exceed 500'.
2. Place sandbags tightly.
3. Dimension may vary to fit field condition.
4. Sandbag barrier shall be a minimum of 3 bags high.
5. The end of the barrier shall be turned up slope.
6. Cross barriers shall be a min of 1/2 and a max of 2/3 the height of the linear barrier.
7. Sandbag rows and layers shall be staggered to eliminate gaps.


LEGEND
DIRECTION OF FLOW
Setback varies (See note 3)
Sandbag barrier (See note 4)
H
Toe of slope
Slope
B
B
SECTION A–A
Slope
Toe of slope
END DETAIL
SECTION B–B
Sandbags
See note 6
SECTION C–C
Sandbag barrier
C
C
Toe of slope
Slope
CROSS BARRIER DETAIL

Straw Bale Barrier SE-9



Objectives

EC	Erosion Control	☒
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence

SE-5 Fiber Rolls

SE-6 Gravel Bag Berm

SE-8 Sandbag Barrier

Description and Purpose

A straw bale barrier is a series of straw bales placed on a level contour to intercept sheet flows. Straw bale barriers pond sheet- flow runoff, allowing sediment to settle out.

Suitable Applications

Straw bale barriers may be suitable:

- As a linear sediment control measure:
  - Below the toe of slopes and erodible slopes
  - As sediment traps at culvert/pipe outlets
  - Below other small cleared areas
  - Along the perimeter of a site
  - Down slope of exposed soil areas
  - Around temporary stockpiles and spoil areas
  - Parallel to a roadway to keep sediment off paved areas
  - Along streams and channels
- As linear erosion control measure:
  - Along the face and at grade breaks of exposed and erodible slopes to shorten slope length and spread runoff as sheet flow

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- At the top of slopes to divert runoff away from disturbed slopes
- As check dams across mildly sloped construction roads

Limitations

Straw bale barriers:

- Are not to be used for extended periods of time because they tend to rot and fall apart
- Are suitable only for sheet flow on slopes of 10 % or flatter
- Are not appropriate for large drainage areas, limit to one acre or less
- May require constant maintenance due to rotting
- Are not recommended for concentrated flow, inlet protection, channel flow, and live streams
- Cannot be made of bale bindings of jute or cotton
- Require labor-intensive installation and maintenance
- Cannot be used on paved surfaces
- Should not to be used for drain inlet protection
- Should not be used on lined ditches
- May introduce undesirable non-native plants to the area

Implementation

General

A straw bale barrier consists of a row of straw bales placed on a level contour. When appropriately placed, a straw bale barrier intercepts and slows sheet flow runoff, causing temporary ponding. The temporary ponding provides quiescent conditions allowing sediment to settle. Straw bale barriers also interrupt the slope length and thereby reduce erosion by reducing the tendency of sheet flows to concentrate into rivulets, which erode rills, and ultimately gullies, into disturbed, sloped soils.

Straw bale barriers have not been as effective as expected due to improper use. These barriers have been placed in streams and drainage ways where runoff volumes and velocities have caused the barriers to wash out. In addition, failure to stake and entrench the straw bale has allowed undercutting and end flow. Use of straw bale barriers in accordance with this BMP should produce acceptable results.

Design and Layout

- Locate straw bale barriers on a level contour.
  - Slopes up to 10:1 (H:V): Straw bales should be placed at a maximum interval of 50 ft (a closer spacing is more effective), with the first row near the toe of slope.
  - Slopes greater than 10:1 (H:V): Not recommended.

- Turn the ends of the straw bale barrier up slope to prevent runoff from going around the barrier.
- Allow sufficient space up slope from the barrier to allow ponding, and to provide room for sediment storage.
- For installation near the toe of the slope, consider moving the barrier away from the slope toe to facilitate cleaning. To prevent flow behind the barrier, sand bags can be placed perpendicular to the barrier to serve as cross barriers.
- Drainage area should not exceed 1 acre, or 0.25 acre per 100 ft of barrier.
- Maximum flow path to the barrier should be limited to 100 ft.
- Straw bale barriers should consist of two parallel rows.
  - Butt ends of bales tightly
  - Stagger butt joints between front and back row
  - Each row of bales must be trenched in and firmly staked
- Straw bale barriers are limited in height to one bale laid on its side.
- Anchor bales with either two wood stakes or four bars driven through the bale and into the soil. Drive the first stake towards the butt joint with the adjacent bale to force the bales together.
- See attached figure for installation details.

Materials

- ***Straw Bale Size:*** Each straw bale should be a minimum of 14 in. wide, 18 in. in height, 36 in. in length and should have a minimum mass of 50 lbs. The straw bale should be composed entirely of vegetative matter, except for the binding material.
- ***Bale Bindings:*** Bales should be bound by steel wire, nylon or polypropylene string placed horizontally. Jute and cotton binding should not be used. Baling wire should be a minimum diameter of 14 gauge. Nylon or polypropylene string should be approximately 12 gauge in diameter with a breaking strength of 80 lbs force.
- ***Stakes:*** Wood stakes should be commercial quality lumber of the size and shape shown on the plans. Each stake should be free from decay, splits or cracks longer than the thickness of the stake, or other defects that would weaken the stakes and cause the stakes to be structurally unsuitable. Steel bar reinforcement should be equal to a #4 designation or greater. End protection should be provided for any exposed bar reinforcement.

Costs

Straw bales cost $5 - $7 each. Adequate labor should be budgeted for installation and maintenance.

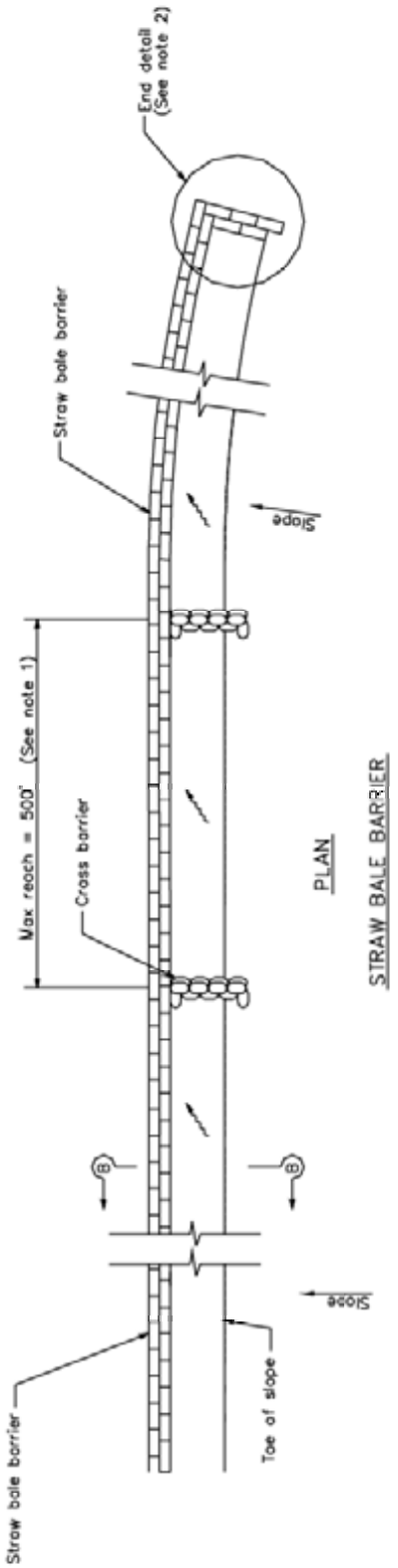
Inspection and Maintenance

Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.
- Straw bales degrade, especially when exposed to moisture. Rotting bales will need to be replaced on a regular basis.
- Replace or repair damaged bales as needed.
- Repair washouts or other damages as needed.
- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site or disposed at an appropriate location.
- Remove straw bales when no longer needed. Remove sediment accumulation, and clean, re-grade, and stabilize the area. Removed sediment should be incorporated in the project or disposed of.

References

Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.



PLAN

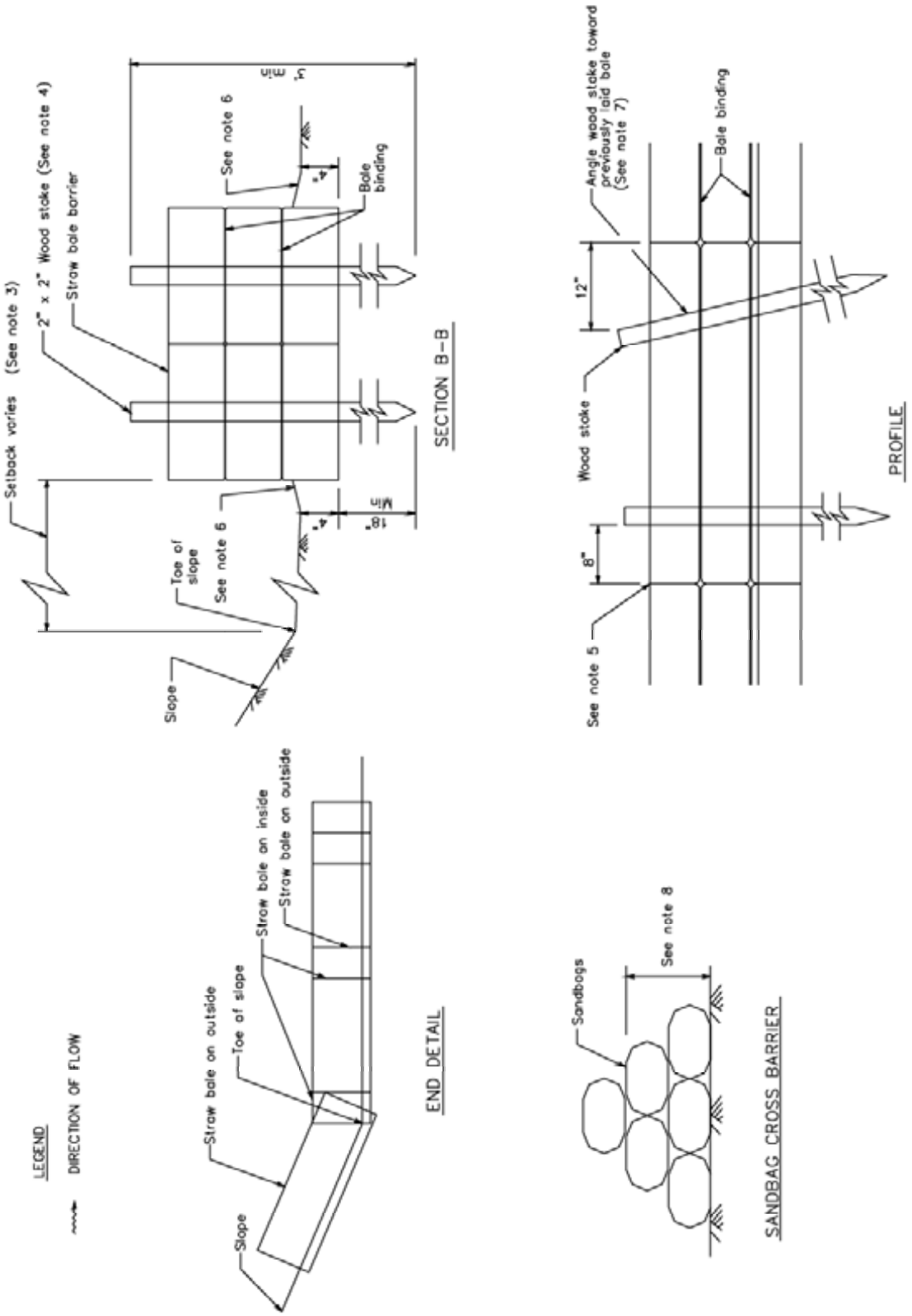
STRAW BALE BARRIER

LEGEND

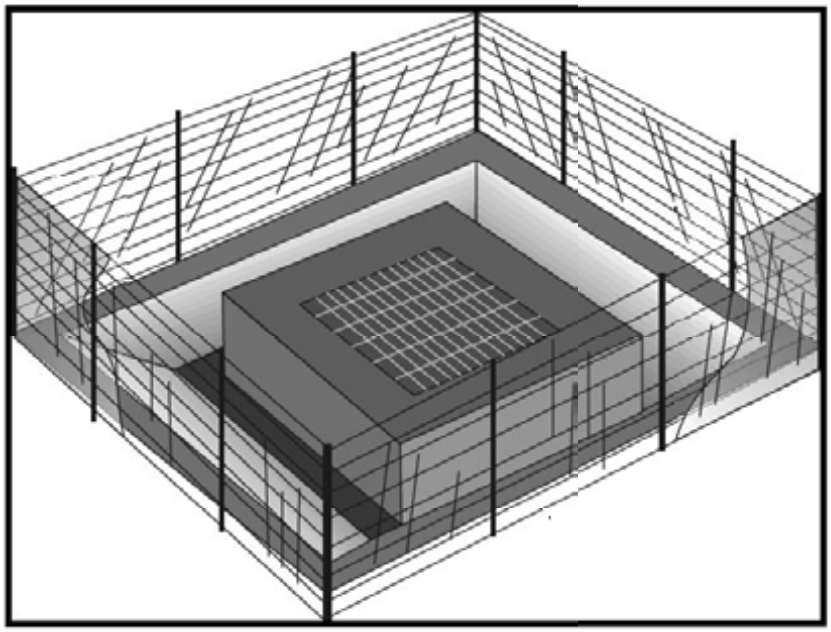
DIRECTION OF FLOW

NOTES

1. Construct the length of each reach so that the change in base elevation along the reach does not exceed 1/2 the height of the linear barrier. In no case shall the reach length exceed 500'.
2. The end of barrier shall be turned up slope.
3. Dimension may vary to fit field condition.
4. Stake dimensions are nominal.
5. Place straw bales tightly together.
6. Tamp embedment spoils against sides of installed bales.
7. Drive angled wood stake before vertical stake to ensure tight abutment to adjacent bale.
8. Sandbag cross barriers should be a min of 1/2 and a max of 2/3 the height of the linear barrier.
9. Sandbag rows and layers should be offset to eliminate gaps.


Setback varies (See note 3)
2" x 2" Wood stake (See note 4)
Straw bale barrier
See note 6
Bale binding
Min. 4"
Min. 18"
Toe of slope
See note 6
Slope
SECTION B-B
Angle wood stake toward previously laid bale (See note 7)
Bale binding
12"
Wood stake
8"
See note 5
PROFILE
LEGEND
DIRECTION OF FLOW
Straw bale on outside
Toe of slope
Straw bale on inside
Straw bale on outside
Slope
END DETAIL
Sandbags
See note 8
SANDBAG CROSS BARRIER

Storm Drain Inlet Protection SE-10



Objectives		
EC	Erosion Control	
SE	Sediment Control	☑
TR	Tracking Control	
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:
☑ **Primary Objective**
☒ **Secondary Objective**

Targeted Constituents	
Sediment	☑
Nutrients	
Trash	☑
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

SE-1 Silt Fence
SE-5 Fiber Rolls
SE-6 Gravel Bag Berm
SE-8 Sandbag Barrier
SE-9 Straw Bale Barrier

Description and Purpose

Storm drain inlet protection consists of a sediment filter or an impounding area around or upstream of a storm drain, drop inlet, or curb inlet. Storm drain inlet protection measures temporarily pond runoff before it enters the storm drain, allowing sediment to settle. Some filter configurations also remove sediment by filtering, but usually the ponding action results in the greatest sediment reduction.

Suitable Applications

Every storm drain inlet receiving sediment-laden runoff should be protected.

Limitations

- Drainage area should not exceed 1 acre.
- Straw bales, while potentially effective, have not produced in practice satisfactory results, primarily due to improper installation.
- Requires an adequate area for water to pond without encroaching into portions of the roadway subject to traffic.
- Inlet protection usually requires other methods of temporary protection to prevent sediment-laden stormwater and non-stormwater discharges from entering the storm drain system.
- Sediment removal may be difficult in high flow conditions or if runoff is heavily sediment laden. If high flow conditions are

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expected, use other onsite sediment trapping techniques in conjunction with inlet protection.

- Frequent maintenance is required.
- For drainage areas larger than 1 acre, runoff should be routed to a sediment-trapping device designed for larger flows. See BMPs SE-2, Sediment Basin, and SE-3, Sediment Traps.
- Excavated drop inlet sediment traps are appropriate where relatively heavy flows are expected, and overflow capability is needed.

Implementation

General

Large amounts of sediment may enter the storm drain system when storm drains are installed before the upslope drainage area is stabilized, or where construction is adjacent to an existing storm drain. In cases of extreme sediment loading, the storm drain itself may clog and lose a major portion of its capacity. To avoid these problems, it is necessary to prevent sediment from entering the system at the inlets.

Inlet control measures presented in this handbook should not be used for inlets draining more than one acre. Runoff from larger disturbed areas should be first routed through SE-2, Sediment Basin or SE-3, Sediment Trap. Different types of inlet protection are appropriate for different applications depending on site conditions and the type of inlet. Inlet protection methods not presented in this handbook should be approved by the local stormwater management agency.

Design and Layout

Identify existing and planned storm drain inlets that have the potential to receive sediment-laden surface runoff. Determine if storm drain inlet protection is needed and which method to use.

- Limit upstream drainage area to 1 acre maximum. For larger drainage areas, use SE-2, Sediment Basin, or SE-3, Sediment Trap, upstream of the inlet protection device.
- The key to successful and safe use of storm drain inlet protection devices is to know where runoff will pond or be diverted.
  - Determine the acceptable location and extent of ponding in the vicinity of the drain inlet. The acceptable location and extent of ponding will influence the type and design of the storm drain inlet protection device.
  - Determine the extent of potential runoff diversion caused by the storm drain inlet protection device. Runoff ponded by inlet protection devices may flow around the device and towards the next downstream inlet. In some cases, this is acceptable; in other cases, serious erosion or downstream property damage can be caused by these diversions. The possibility of runoff diversions will influence whether or not storm drain inlet protection is suitable; and, if suitable, the type and design of the device.
- The location and extent of ponding, and the extent of diversion, can usually be controlled through appropriate placement of the inlet protection device. In some cases, moving the

inlet protection device a short distance upstream of the actual inlet can provide more efficient sediment control, limit ponding to desired areas, and prevent or control diversions.

- Four types of inlet protection are presented below. However, it is recognized that other effective methods and proprietary devices exist and may be selected.
  - Filter Fabric Fence: Appropriate for drainage basins with less than a 5% slope, sheet flows, and flows under 0.5 cfs.
  - Excavated Drop Inlet Sediment Trap An excavated area around the inlet to trap sediment (SE-3).
  - Gravel bag barrier: Used to create a small sediment trap upstream of inlets on sloped, paved streets. Appropriate for sheet flow or when concentrated flow may exceed 0.5 cfs, and where overtopping is required to prevent flooding.
  - Block and Gravel Filter: Appropriate for flows greater than 0.5 cfs.
- Select the appropriate type of inlet protection and design as referred to or as described in this fact sheet.
- Provide area around the inlet for water to pond without flooding structures and property.
- Grates and spaces around all inlets should be sealed to prevent seepage of sediment-laden water.
- Excavate sediment sumps (where needed) 1 to 2 ft with 2:1 side slopes around the inlet.

Installation

- ***DI Protection Type 1 - Filter Fabric Fence*** - The filter fabric fence (Type 1) protection is shown in the attached figure. Similar to constructing a silt fence; see BMP SE-1, Silt Fence. Do not place filter fabric underneath the inlet grate since the collected sediment may fall into the drain inlet when the fabric is removed or replaced.
  1. Excavate a trench approximately 6 in. wide and 6 in. deep along the line of the silt fence inlet protection device.
  2. Place 2 in. by 2 in. wooden stakes around the perimeter of the inlet a maximum of 3 ft apart and drive them at least 18 in. into the ground or 12 in. below the bottom of the trench. The stakes must be at least 43 in.
  3. Lay fabric along bottom of trench, up side of trench, and then up stakes. See SE-1, Silt Fence, for details. The maximum silt fence height around the inlet is 24 in.
  4. Staple the filter fabric (for materials and specifications, see SE-1, Silt Fence) to wooden stakes. Use heavy-duty wire staples at least 1 in. in length.
  5. Backfill the trench with gravel or compacted earth all the way around.
- ***DI Protection Type 2 - Excavated Drop Inlet Sediment Trap*** - The excavated drop inlet sediment trap (Type 2) is shown in the attached figures. Install filter fabric fence in

accordance with DI Protection Type 1. Size excavated trap to provide a minimum storage capacity calculated at the rate 67 yd^3/acre of drainage area.

- ***DI Protection Type 3 - Gravel bag*** - The gravel bag barrier (Type 3) is shown in the figures. Flow from a severe storm should not overtop the curb. In areas of high clay and silts, use filter fabric and gravel as additional filter media. Construct gravel bags in accordance with SE-6, Gravel Bag Berm. Gravel bags should be used due to their high permeability.

  1. Use sand bag made of geotextile fabric (not burlap) and fill with 0.75 in. rock or 0.25 in. pea gravel.
  2. Construct on gently sloping street.
  3. Leave room upstream of barrier for water to pond and sediment to settle.
  4. Place several layers of sand bags – overlapping the bags and packing them tightly together.
  5. Leave gap of one bag on the top row to serve as a spillway. Flow from a severe storm (e.g., 10 year storm) should not overtop the curb.

- ***DI Protection Type 4 – Block and Gravel Filter*** - The block and gravel filter (Type 4) is shown in the figures. Block and gravel filters are suitable for curb inlets commonly used in residential, commercial, and industrial construction.

  1. Place hardware cloth or comparable wire mesh with 0.5 in. openings over the drop inlet so that the wire extends a minimum of 1 ft beyond each side of the inlet structure. If more than one strip is necessary, overlap the strips. Place filter fabric over the wire mesh.
  2. Place concrete blocks lengthwise on their sides in a single row around the perimeter of the inlet, so that the open ends face outward, not upward. The ends of adjacent blocks should abut. The height of the barrier can be varied, depending on design needs, by stacking combinations of blocks that are 4 in., 8 in., and 12 in. wide. The row of blocks should be at least 12 in. but no greater than 24 in. high.
  3. Place wire mesh over the outside vertical face (open end) of the concrete blocks to prevent stone from being washed through the blocks. Use hardware cloth or comparable wire mesh with 0.5 in. opening.
  4. Pile washed stone against the wire mesh to the top of the blocks. Use 0.75 to 3 in.

Costs

- Average annual cost for installation and maintenance (one year useful life) is $200 per inlet.

Inspection and Maintenance

- Inspect BMPs prior to forecast rain, daily during extended rain events, after rain events, weekly during the rainy season, and at two-week intervals during the non-rainy season.

- Filter Fabric Fences. If the fabric becomes clogged, torn, or degrades, it should be replaced. Make sure the stakes are securely driven in the ground and are in good shape (i.e., not bent, cracked, or splintered, and are reasonably perpendicular to the ground). Replace damaged stakes.

- Gravel Filters. If the gravel becomes clogged with sediment, it must be carefully removed from the inlet and either cleaned or replaced. Since cleaning gravel at a construction site may be difficult, consider using the sediment-laden stone as fill material and put fresh stone around the inlet. Inspect bags for holes, gashes, and snags, and replace bags as needed. Check gravel bags for proper arrangement and displacement.

- Sediment that accumulates in the BMP must be periodically removed in order to maintain BMP effectiveness. Sediment should be removed when the sediment accumulation reaches one-third of the barrier height. Sediment removed during maintenance may be incorporated into earthwork on the site ore disposed at an appropriate location.

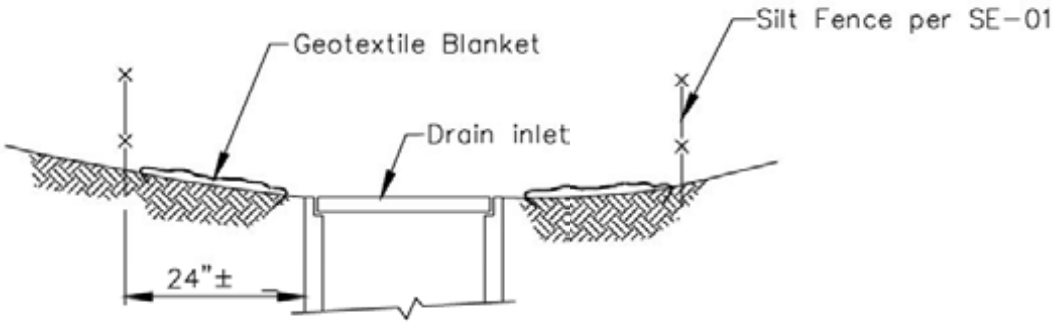
- Remove storm drain inlet protection once the drainage area is stabilized.

  - Clean and regrade area around the inlet and clean the inside of the storm drain inlet as it must be free of sediment and debris at the time of final inspection.

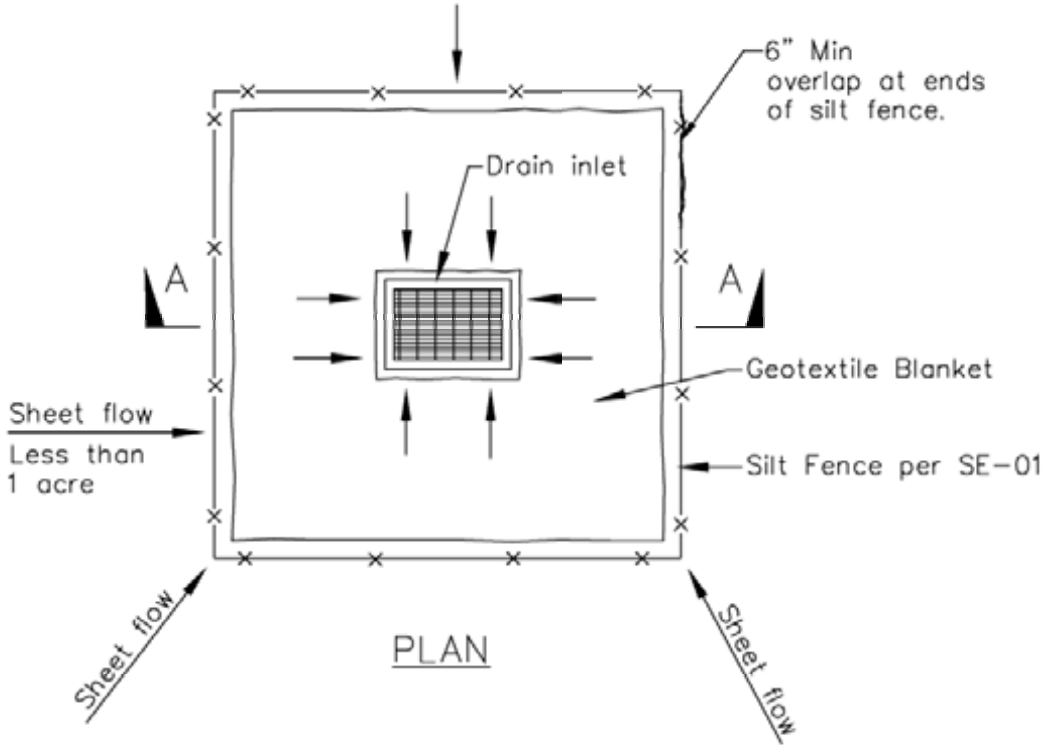
References

Stormwater Quality Handbooks - Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stormwater Management Manual for The Puget Sound Basin, Washington State Department of Ecology, Public Review Draft, 1991.



SECTION A–A

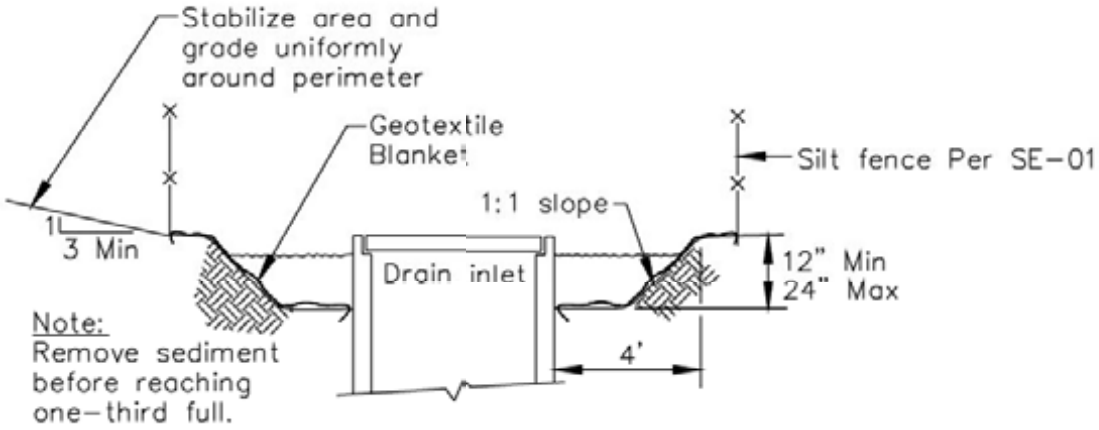


PLAN

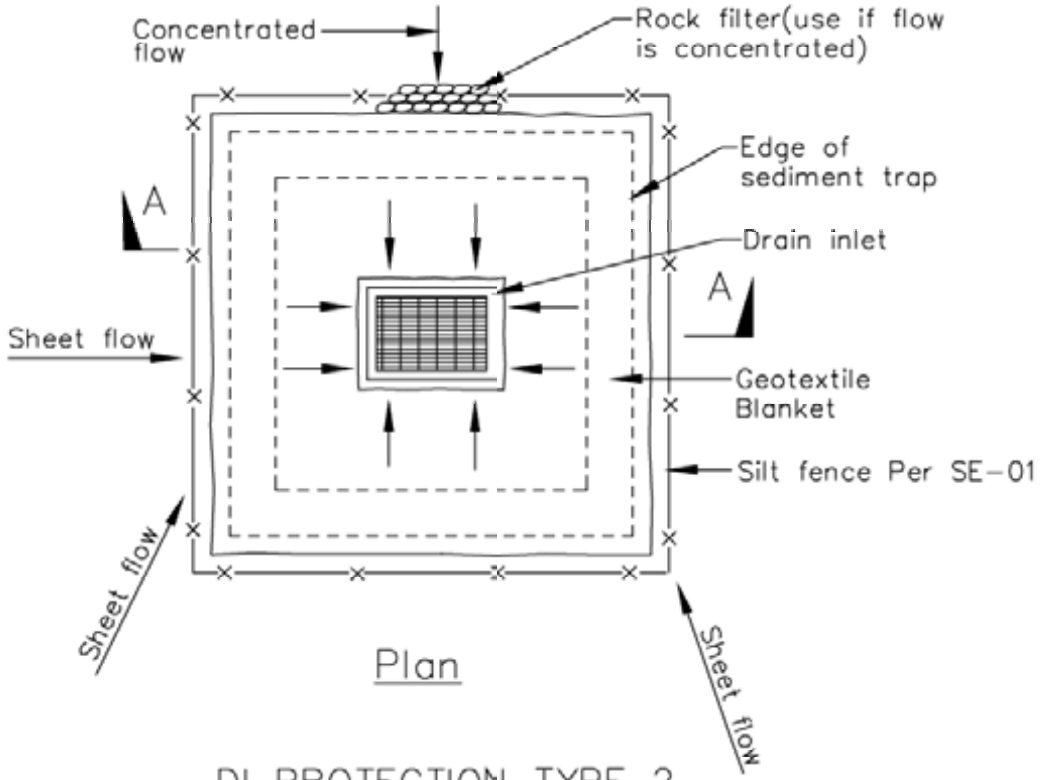
DI PROTECTION TYPE 1
NOT TO SCALE

NOTES:

1. For use in areas where grading has been completed and final soil stabilization and seeding are pending.
2. Not applicable in paved areas.
3. Not applicable with concentrated flows.



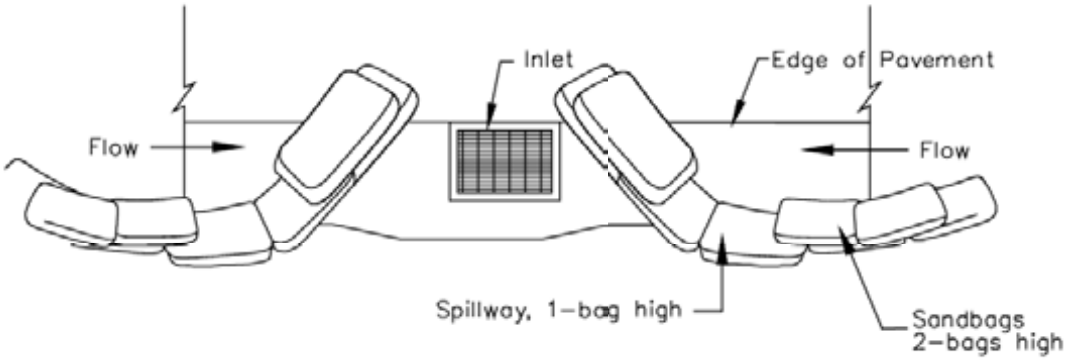
Section A-A



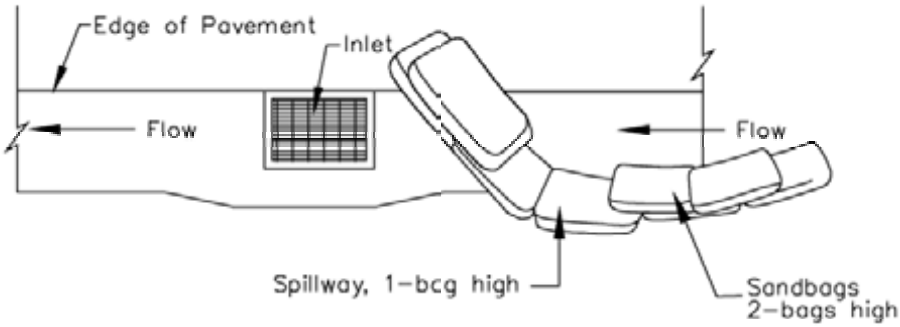
Plan

DI PROTECTION TYPE 2
NOT TO SCALE

Notes
1. For use in cleared and grubbed and in graded areas.
2. Shape basin so that longest inflow area faces longest length of trap.
3. For concentrated flows, shape basin in 2:1 ratio with length oriented towards direction of flow.



TYPICAL PROTECTION FOR INLET ON SUMP

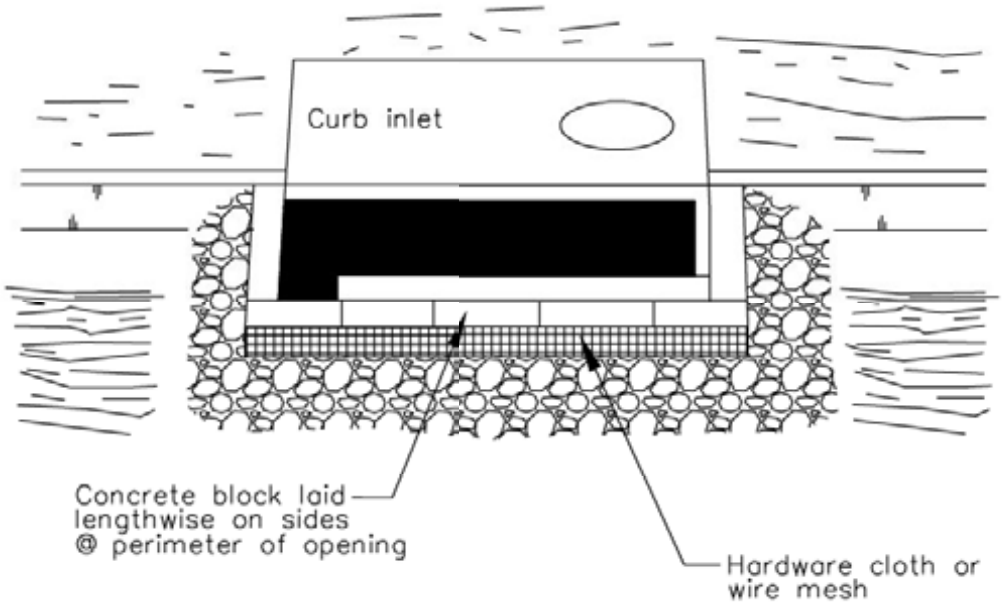


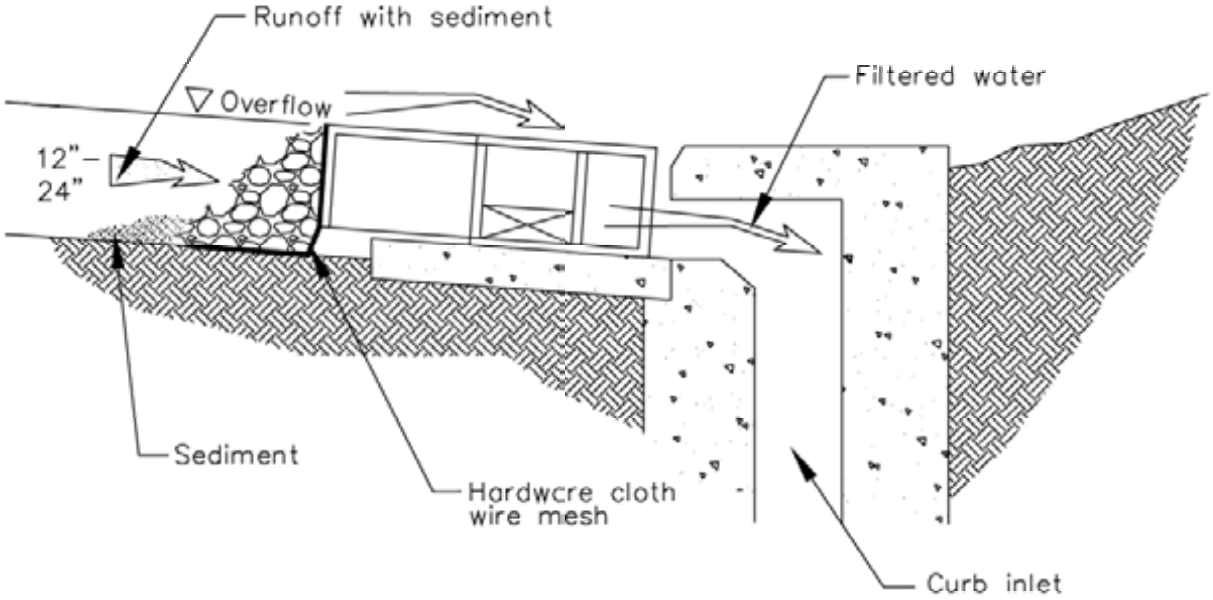
TYPICAL PROTECTION FOR INLET ON GRADE

NOTES:

1. Intended for short-term use.
2. Use to inhibit non-storm water flow.
3. Allow for proper maintenance and cleanup.
4. Bags must be removed after adjacent operation is completed
5. Not applicable in areas with high silts and clays without filter fabric.

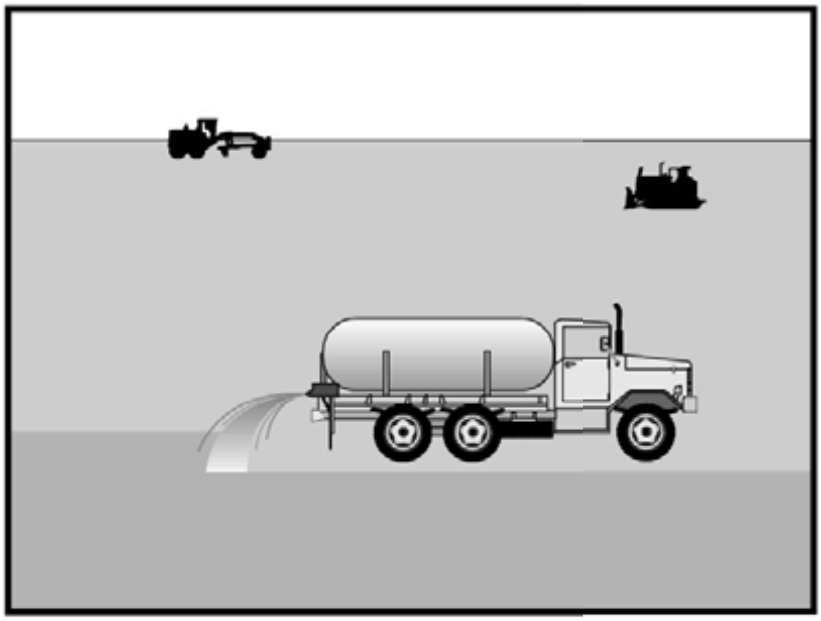
DI PROTECTION TYPE 3
NOT TO SCALE





DI PROTECTION – TYPE 4
NOT TO SCALE

Wind Erosion Control WE-1



Objectives

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	
WE	Wind Erosion Control	☑
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

Wind erosion or dust control consists of applying water or other dust palliatives as necessary to prevent or alleviate dust nuisance generated by construction activities. Covering small stockpiles or areas is an alternative to applying water or other dust palliatives.

Suitable Applications

Wind erosion control BMPs are suitable during the following construction activities:

- Construction vehicle traffic on unpaved roads
- Drilling and blasting activities
- Sediment tracking onto paved roads
- Soils and debris storage piles
- Batch drop from front-end loaders
- Areas with unstabilized soil
- Final grading/site stabilization

Limitations

- Watering prevents dust only for a short period and should be applied daily (or more often) to be effective.
- Over watering may cause erosion.

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- Oil or oil-treated subgrade should not be used for dust control because the oil may migrate into drainageways and/or seep into the soil.
- Effectiveness depends on soil, temperature, humidity, and wind velocity.
- Chemically treated sub grades may make the soil water repellant, interfering with long-term infiltration and the vegetation/re-vegetation of the site. Some chemical dust suppressants may be subject to freezing and may contain solvents and should be handled properly.
- Asphalt, as a mulch tack or chemical mulch, requires a 24-hour curing time to avoid adherence to equipment, worker shoes, etc. Application should be limited because asphalt surfacing may eventually migrate into the drainage system.
- In compacted areas, watering and other liquid dust control measures may wash sediment or other constituents into the drainage system.

Implementation

General

California's Mediterranean climate, with short wet seasons and long hot dry seasons, allows the soils to thoroughly dry out. During these dry seasons, construction activities are at their peak, and disturbed and exposed areas are increasingly subject to wind erosion, sediment tracking and dust generated by construction equipment.

Dust control, as a BMP, is a practice that is already in place for many construction activities. Los Angeles, the North Coast, and Sacramento, among others, have enacted dust control ordinances for construction activities that cause dust to be transported beyond the construction project property line.

Recently, the State Air Resources Control Board has, under the authority of the Clean Air Act, started to address air quality in relation to inhalable particulate matter less than 10 microns (PM-10). Approximately 90 percent of these small particles are considered to be dust. Existing dust control regulations by local agencies, municipal departments, public works department, and public health departments are in place in some regions within California.

Many local agencies require dust control in order to comply with local nuisance laws, opacity laws (visibility impairment) and the requirements of the Clean Air Act. The following are measures that local agencies may have already implemented as requirements for dust control from contractors:

- Construction and Grading Permits: Require provisions for dust control plans.
- Opacity Emission Limits: Enforce compliance with California air pollution control laws.
- Increase Overall Enforcement Activities: Priority given to cases involving citizen complaints.
- Maintain Field Application Records: Require records of dust control measures from contractor;
- Stormwater Pollution Prevention Plan: (SWPPP): Integrate dust control measures into SWPPP.

Dust Control Practices

Dust control BMPs generally stabilize exposed surfaces and minimize activities that suspend or track dust particles. The following table shows dust control practices that can be applied to site conditions that cause dust. For heavily traveled and disturbed areas, wet suppression (watering), chemical dust suppression, gravel asphalt surfacing, temporary gravel construction entrances, equipment wash-out areas, and haul truck covers can be employed as dust control applications. Permanent or temporary vegetation and mulching can be employed for areas of occasional or no construction traffic. Preventive measures would include minimizing surface areas to be disturbed, limiting onsite vehicle traffic to 15 mph, and controlling the number and activity of vehicles on a site at any given time.

SITE CONDITION	DUST CONTROL PRACTICES								
	Permanent Vegetation	Mulching	Wet Suppression (Watering)	Chemical Dust Suppression	Gravel or Asphalt	Silt Fences	Temporary Gravel Construction Entrances/Equipment Wash Down	Haul Truck Covers	Minimize Extent of Disturbed Area
Disturbed Areas not Subject to Traffic	X	X	X	X	X				X
Disturbed Areas Subject to Traffic			X	X	X		X		X
Material Stock Pile Stabilization			X	X		X			X
Demolition			X				X	X	
Clearing/ Excavation			X	X		X			X
Truck Traffic on Unpaved Roads			X	X	X		X	X	
Mud/Dirt Carry Out					X		X		

Additional preventive measures include:

- Schedule construction activities to minimize exposed area (EC-1, Scheduling).
- Quickly stabilize exposed soils using vegetation, mulching, spray-on adhesives, calcium chloride, sprinkling, and stone/gravel layering.
- Identify and stabilize key access points prior to commencement of construction.
- Minimize the impact of dust by anticipating the direction of prevailing winds.
- Direct most construction traffic to stabilized roadways within the project site.
- Water should be applied by means of pressure-type distributors or pipelines equipped with a spray system or hoses and nozzles that will ensure even distribution.
- All distribution equipment should be equipped with a positive means of shutoff.
- Unless water is applied by means of pipelines, at least one mobile unit should be available at all times to apply water or dust palliative to the project.

- If reclaimed waste water is used, the sources and discharge must meet California Department of Health Services water reclamation criteria and the Regional Water Quality Control Board requirements. Non-potable water should not be conveyed in tanks or drain pipes that will be used to convey potable water and there should be no connection between potable and non-potable supplies. Non-potable tanks, pipes, and other conveyances should be marked, "NON-POTABLE WATER - DO NOT DRINK."
- Materials applied as temporary soil stabilizers and soil binders also generally provide wind erosion control benefits.
- Pave or chemically stabilize access points where unpaved traffic surfaces adjoin paved roads.
- Provide covers for haul trucks transporting materials that contribute to dust.
- Provide for wet suppression or chemical stabilization of exposed soils.
- Provide for rapid clean up of sediments deposited on paved roads. Furnish stabilized construction road entrances and vehicle wash down areas.
- Stabilize inactive construction sites using vegetation or chemical stabilization methods.
- Limit the amount of areas disturbed by clearing and earth moving operations by scheduling these activities in phases.

For chemical stabilization, there are many products available for chemically stabilizing gravel roadways and stockpiles. If chemical stabilization is used, the chemicals should not create any adverse effects on stormwater, plant life, or groundwater.

Costs

Installation costs for water and chemical dust suppression are low, but annual costs may be quite high since these measures are effective for only a few hours to a few days.

Inspection and Maintenance

- Inspect and verify that activity-based BMPs are in place prior to the commencement of associated activities. While activities associated with the BMP are under way, inspect weekly during the rainy season and at two-week intervals in the non-rainy season to verify continued BMP implementation.
- Check areas protected to ensure coverage.
- Most dust control measures require frequent, often daily, or multiple times per day attention.

References

Best Management Practices and Erosion Control Manual for Construction Sites, Flood Control District of Maricopa County, Arizona, September 1992.

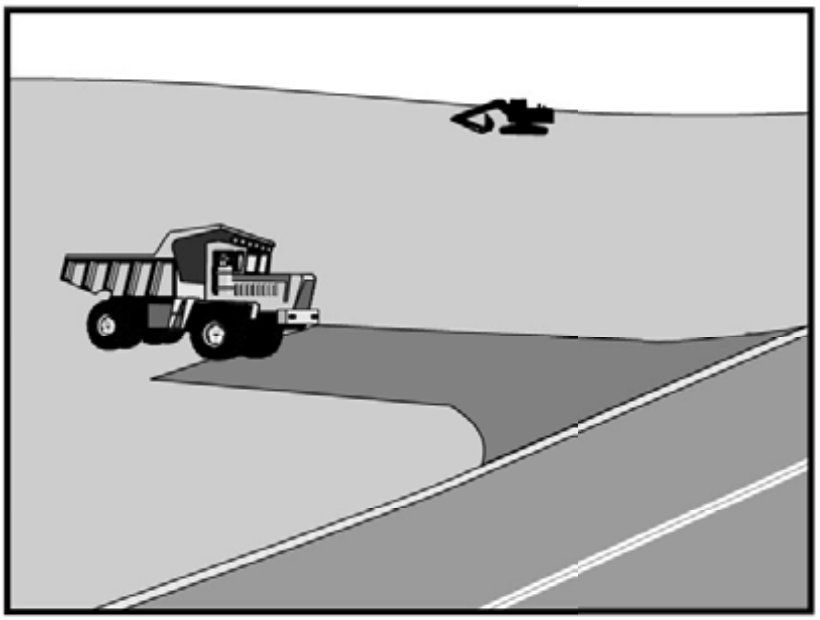
California Air Pollution Control Laws, California Air Resources Board, 1992.

Caltrans, Standard Specifications, Sections 10, "Dust Control"; Section 17, "Watering"; and Section 18, "Dust Palliative".

Prospects for Attaining the State Ambient Air Quality Standards for Suspended Particulate Matter (PM10), Visibility Reducing Particles, Sulfates, Lead, and Hydrogen Sulfide, California Air Resources Board, April 1991.

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

Stabilized Construction Entrance/Exit TC-1



Description and Purpose

A stabilized construction access is defined by a point of entrance/exit to a construction site that is stabilized to reduce the tracking of mud and dirt onto public roads by construction vehicles.

Suitable Applications

Use at construction sites:

- Where dirt or mud can be tracked onto public roads.
- Adjacent to water bodies.
- Where poor soils are encountered.
- Where dust is a problem during dry weather conditions.

Limitations

- Entrances and exits require periodic top dressing with additional stones.
- This BMP should be used in conjunction with street sweeping on adjacent public right of way.
- Entrances and exits should be constructed on level ground only.
- Stabilized construction entrances are rather expensive to construct and when a wash rack is included, a sediment trap of some kind must also be provided to collect wash water runoff.

Objectives

EC	Erosion Control	☒
SE	Sediment Control	☒
TC	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

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Stabilized Construction Entrance/Exit TC-1

Implementation

General

A stabilized construction entrance is a pad of aggregate underlain with filter cloth located at any point where traffic will be entering or leaving a construction site to or from a public right of way, street, alley, sidewalk, or parking area. The purpose of a stabilized construction entrance is to reduce or eliminate the tracking of sediment onto public rights of way or streets. Reducing tracking of sediments and other pollutants onto paved roads helps prevent deposition of sediments into local storm drains and production of airborne dust.

Where traffic will be entering or leaving the construction site, a stabilized construction entrance should be used. NPDES permits require that appropriate measures be implemented to prevent tracking of sediments onto paved roadways, where a significant source of sediments is derived from mud and dirt carried out from unpaved roads and construction sites.

Stabilized construction entrances are moderately effective in removing sediment from equipment leaving a construction site. The entrance should be built on level ground. Advantages of the Stabilized Construction Entrance/Exit is that it does remove some sediment from equipment and serves to channel construction traffic in and out of the site at specified locations. Efficiency is greatly increased when a washing rack is included as part of a stabilized construction entrance/exit.

Design and Layout

- Construct on level ground where possible.
- Select 3 to 6 in. diameter stones.
- Use minimum depth of stones of 12 in. or as recommended by soils engineer.
- Construct length of 50 ft minimum, and 30 ft minimum width.
- Rumble racks constructed of steel panels with ridges and installed in the stabilized entrance/exit will help remove additional sediment and to keep adjacent streets clean.
- Provide ample turning radii as part of the entrance.
- Limit the points of entrance/exit to the construction site.
- Limit speed of vehicles to control dust.
- Properly grade each construction entrance/exit to prevent runoff from leaving the construction site.
- Route runoff from stabilized entrances/exits through a sediment trapping device before discharge.
- Design stabilized entrance/exit to support heaviest vehicles and equipment that will use it.
- Select construction access stabilization (aggregate, asphaltic concrete, concrete) based on longevity, required performance, and site conditions. Do not use asphalt concrete (AC) grindings for stabilized construction access/roadway.

- If aggregate is selected, place crushed aggregate over geotextile fabric to at least 12 in. depth, or place aggregate to a depth recommended by a geotechnical engineer. A crushed aggregate greater than 3 in. but smaller than 6 in. should be used.
- Designate combination or single purpose entrances and exits to the construction site.
- Require that all employees, subcontractors, and suppliers utilize the stabilized construction access.
- Implement SE-7, Street Sweeping and Vacuuming, as needed.
- All exit locations intended to be used for more than a two-week period should have stabilized construction entrance/exit BMPs.

Inspection and Maintenance

- Inspect and verify that activity–based BMPs are in place prior to the commencement of associated activities. While activities associated with the BMPs are under way, inspect weekly during the rainy season and of two-week intervals in the non-rainy season to verify continued BMP implementation.
- Inspect local roads adjacent to the site daily. Sweep or vacuum to remove visible accumulated sediment.
- Remove aggregate, separate and dispose of sediment if construction entrance/exit is clogged with sediment.
- Keep all temporary roadway ditches clear.
- Check for damage and repair as needed.
- Replace gravel material when surface voids are visible.
- Remove all sediment deposited on paved roadways within 24 hours.
- Remove gravel and filter fabric at completion of construction.

Costs

Average annual cost for installation and maintenance may vary from $1,200 to $4,800 each, averaging $2,400 per entrance. Costs will increase with addition of washing rack, and sediment trap. With wash rack, costs range from $1,200 - $6,000 each, averaging $3,600 per entrance.

References

Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

National Management Measures to Control Nonpoint Source Pollution from Urban Areas, USEPA Agency, 2002.

Proposed Guidance Specifying Management Measures for Sources of Nonpoint Pollution in Coastal Waters, Work Group Working Paper, USEPA, April 1992.

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

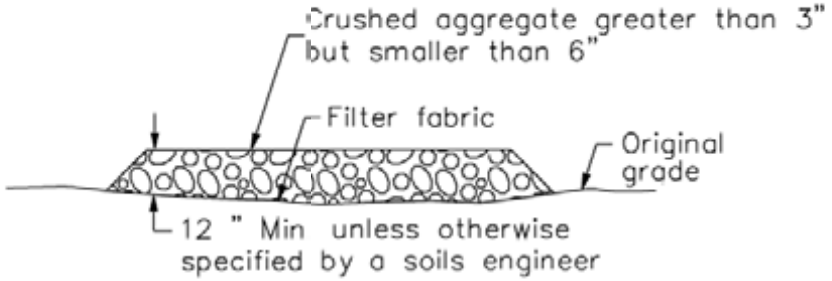
Stormwater Management of the Puget Sound Basin, Technical Manual, Publication #91-75, Washington State Department of Ecology, February 1992.

Virginia Erosion and Sedimentation Control Handbook, Virginia Department of Conservation and Recreation, Division of Soil and Water Conservation, 1991.

Guidance Specifying Management Measures for Nonpoint Pollution in Coastal Waters, EPA 840-B-9-002, USEPA, Office of Water, Washington DC, 1993.

Water Quality Management Plan for the Lake Tahoe Region, Volume II, Handbook of Management Practices, Tahoe Regional Planning Agency, November 1988.

Stabilized Construction Entrance/Exit TC-1



SECTION B–B
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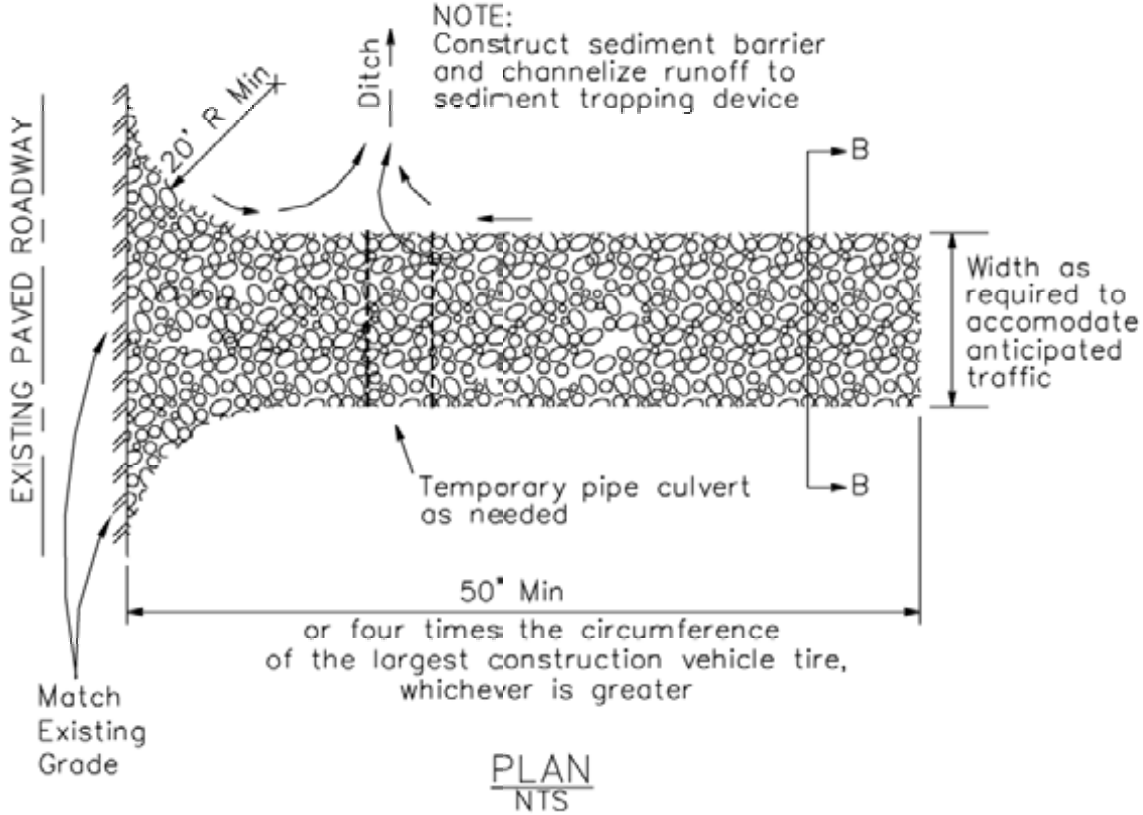
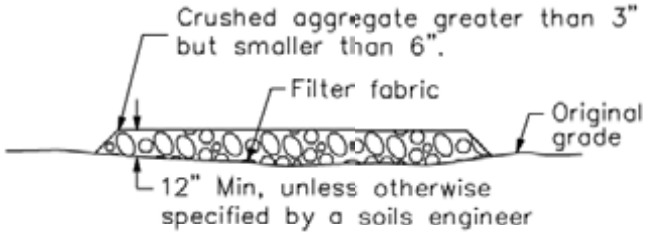


PLAN
NTS

Stabilized Construction Entrance/Exit TC-1



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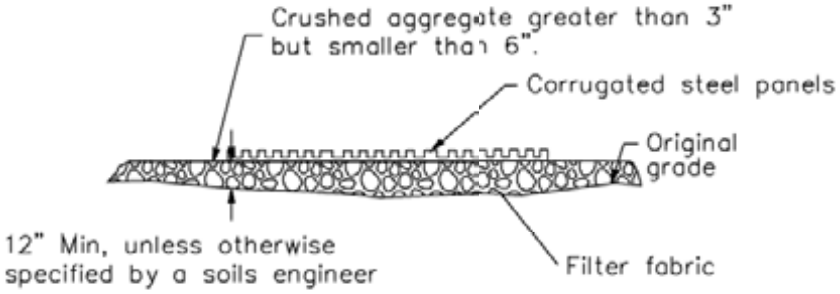

SECTION A–A
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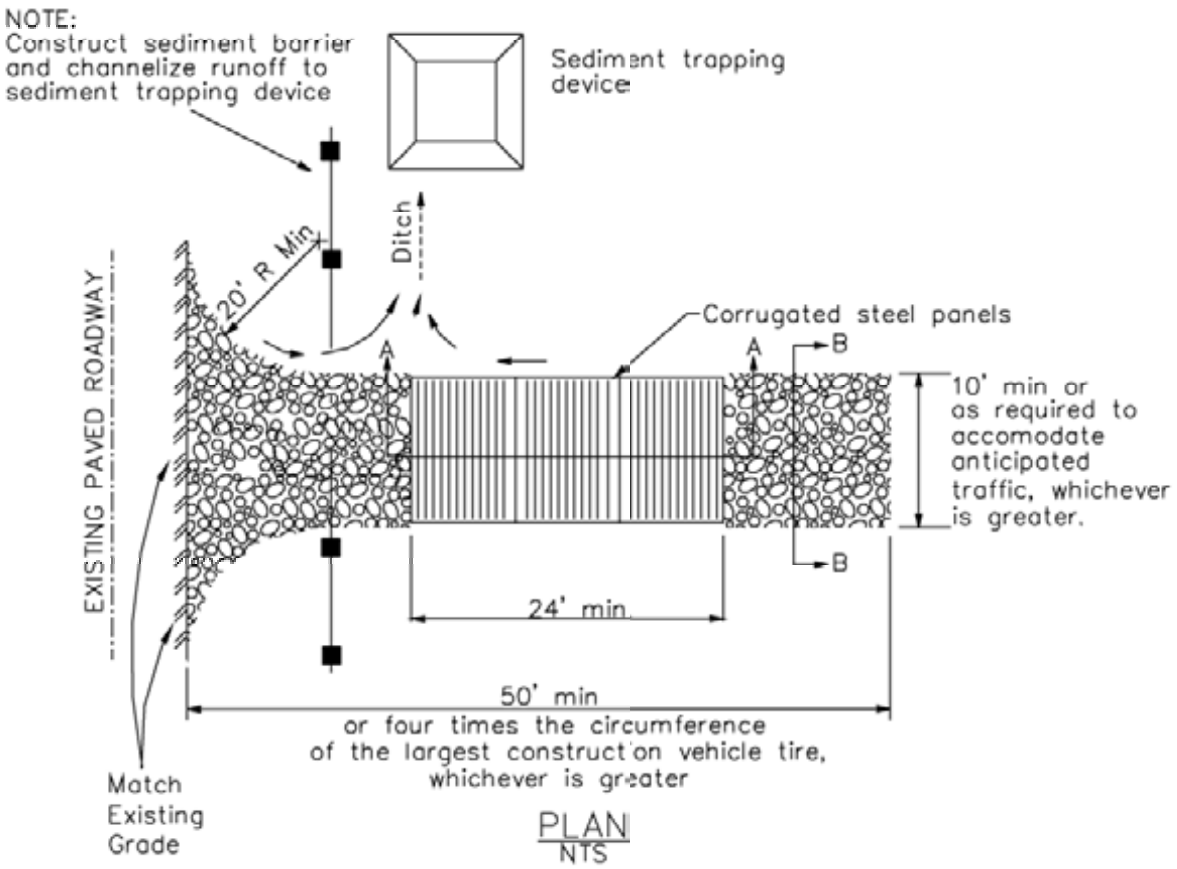

PLAN
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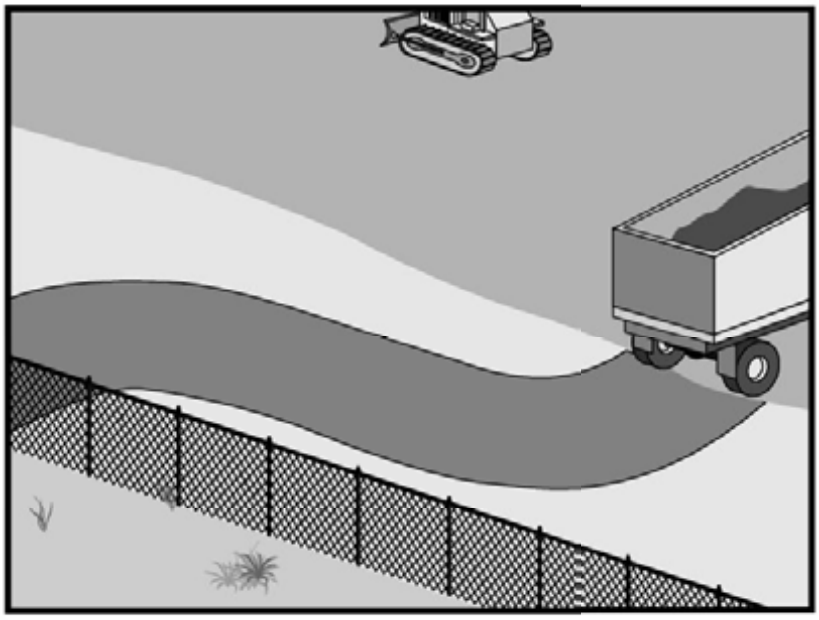
Stabilized Construction Roadway TC-2



Objectives

EC	Erosion Control	☒
SE	Sediment Control	☒
TC	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

None

Description and Purpose

Access roads, subdivision roads, parking areas, and other onsite vehicle transportation routes should be stabilized immediately after grading, and frequently maintained to prevent erosion and control dust.

Suitable Applications

This BMP should be applied for the following conditions:

- Temporary Construction Traffic:
  - Phased construction projects and offsite road access
  - Construction during wet weather
- Construction roadways and detour roads:
  - Where mud tracking is a problem during wet weather
  - Where dust is a problem during dry weather
  - Adjacent to water bodies
  - Where poor soils are encountered

Limitations

- The roadway must be removed or paved when construction is complete.

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- Certain chemical stabilization methods may cause stormwater or soil pollution and should not be used. See WE-1, Wind Erosion Control.
- Management of construction traffic is subject to air quality control measures. Contact the local air quality management agency.
- Materials will likely need to be removed prior to final project grading and stabilization.
- Use of this BMP may not be applicable to very short duration projects.

Implementation

General

Areas that are graded for construction vehicle transport and parking purposes are especially susceptible to erosion and dust. The exposed soil surface is continually disturbed, leaving no opportunity for vegetative stabilization. Such areas also tend to collect and transport runoff waters along their surfaces. During wet weather, they often become muddy quagmires that generate significant quantities of sediment that may pollute nearby streams or be transported offsite on the wheels of construction vehicles. Dirt roads can become so unstable during wet weather that they are virtually unusable.

Efficient construction road stabilization not only reduces onsite erosion but also can significantly speed onsite work, avoid instances of immobilized machinery and delivery vehicles, and generally improve site efficiency and working conditions during adverse weather

Installation/Application Criteria

Permanent roads and parking areas should be paved as soon as possible after grading. As an alternative where construction will be phased, the early application of gravel or chemical stabilization may solve potential erosion and stability problems. Temporary gravel roadway should be considered during the rainy season and on slopes greater than 5%.

Temporary roads should follow the contour of the natural terrain to the maximum extent possible. Slope should not exceed 15%. Roadways should be carefully graded to drain transversely. Provide drainage swales on each side of the roadway in the case of a crowned section or one side in the case of a super elevated section. Simple gravel berms without a trench can also be used.

Installed inlets should be protected to prevent sediment laden water from entering the storm sewer system (SE-10, Storm Drain Inlet Protection). In addition, the following criteria should be considered.

- Road should follow topographic contours to reduce erosion of the roadway.
- The roadway slope should not exceed 15%.
- Chemical stabilizers or water are usually required on gravel or dirt roads to prevent dust (WE-1, Wind Erosion Control).
- Properly grade roadway to prevent runoff from leaving the construction site.
- Design stabilized access to support heaviest vehicles and equipment that will use it.

- Stabilize roadway using aggregate, asphalt concrete, or concrete based on longevity, required performance, and site conditions. The use of cold mix asphalt or asphalt concrete (AC) grindings for stabilized construction roadway is not allowed.
- Coordinate materials with those used for stabilized construction entrance/exit points.
- If aggregate is selected, place crushed aggregate over geotextile fabric to at least 12 in. depth. A crushed aggregate greater than 3 in. but smaller than 6 in. should be used.

Inspection and Maintenance

- Inspect and verify that activity–based BMPs are in place prior to the commencement of associated activities. While activities associated with the BMP are under way, impact weekly during the rainy season and of two-week intervals in the non-rainy season to verify continued BMP implementation.
- Keep all temporary roadway ditches clear.
- When no longer required, remove stabilized construction roadway and re-grade and repair slopes.
- Periodically apply additional aggregate on gravel roads.
- Active dirt construction roads are commonly watered three or more times per day during the dry season.

Costs

Gravel construction roads are moderately expensive, but cost is often balanced by reductions in construction delay. No additional costs for dust control on construction roads should be required above that needed to meet local air quality requirements.

References

Blueprint for a Clean Bay: Best Management Practices to Prevent Stormwater Pollution from Construction Related Activities; Santa Clara Valley Nonpoint Source Pollution Control Program, 1995.

Coastal Nonpoint Pollution Control Program; Program Development and Approval Guidance, Working Group, Working Paper; USEPA, April 1992.

Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

Stormwater Quality Handbooks Construction Site Best Management Practices (BMPs) Manual, State of California Department of Transportation (Caltrans), November 2000.

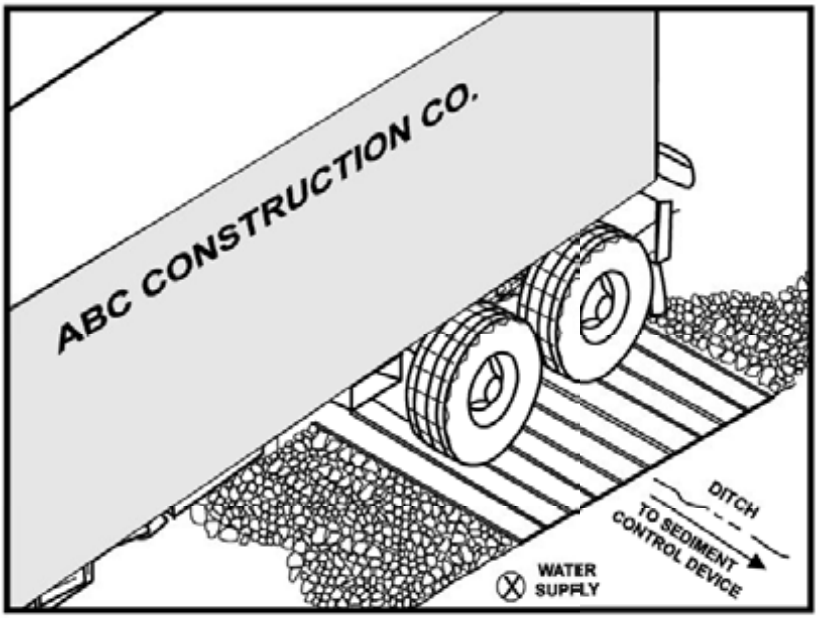
Stormwater Management for Construction Activities, Developing Pollution Prevention Plans and Best Management Practices, EPA 832-R-92005; USEPA, April 1992.

Stormwater Management of the Puget Sound Basin, Technical Manual, Publication #91-75, Washington State Department of Ecology, February 1992.

Virginia Erosion and Sedimentation Control Handbook, Virginia Department of Conservation and Recreation, Division of Soil and Water Conservation, 1991.

Water Quality Management Plan for the Lake Tahoe Region, Volume II, Handbook of Management Practices, Tahoe Regional Planning Agency, November 1988.

Entrance/Outlet Tire Wash TC-3



Objectives

EC	Erosion Control	
SE	Sediment Control	☒
TC	Tracking Control	☑
WE	Wind Erosion Control	
NS	Non-Stormwater Management Control	
WM	Waste Management and Materials Pollution Control	

Legend:

☑ **Primary Objective**

☒ **Secondary Objective**

Targeted Constituents

Sediment	☑
Nutrients	
Trash	
Metals	
Bacteria	
Oil and Grease	
Organics	

Potential Alternatives

TC-1 Stabilized Construction Entrance/Exit

Description and Purpose

A tire wash is an area located at stabilized construction access points to remove sediment from tires and under carriages and to prevent sediment from being transported onto public roadways.

Suitable Applications

Tire washes may be used on construction sites where dirt and mud tracking onto public roads by construction vehicles may occur.

Limitations

- The tire wash requires a supply of wash water.
- A turnout or doublewide exit is required to avoid having entering vehicles drive through the wash area.
- Do not use where wet tire trucks leaving the site leave the road dangerously slick.

Implementation

- Incorporate with a stabilized construction entrance/exit. See TC-1, Stabilized Construction Entrance/Exit.
- Construct on level ground when possible, on a pad of coarse aggregate greater than 3 in. but smaller than 6 in. A geotextile fabric should be placed below the aggregate.
- Wash rack should be designed and constructed/manufactured for anticipated traffic loads.

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- Provide a drainage ditch that will convey the runoff from the wash area to a sediment trapping device. The drainage ditch should be of sufficient grade, width, and depth to carry the wash runoff.
- Use hoses with automatic shutoff nozzles to prevent hoses from being left on.
- Require that all employees, subcontractors, and others that leave the site with mud caked tires and undercarriages to use the wash facility.
- Implement SC-7, Street Sweeping and Vacuuming, as needed.

Costs

Costs are low for installation of wash rack.

Inspection and Maintenance

- Inspect and verify that activity–based BMPs are in place prior to the commencement of associated activities. While activities associated with the BMP are under way, inspect weekly during the rainy season and of two-week intervals in the non-rainy season to verify continued BMP implementation.
- Inspect BMPs subject to non-stormwater discharge daily while non-stormwater discharges occur.
- Remove accumulated sediment in wash rack and/or sediment trap to maintain system performance.
- Inspect routinely for damage and repair as needed.

References

Blueprint for a Clean Bay: Best Management Practices to Prevent Stormwater Pollution from Construction Related Activities; Santa Clara Valley Nonpoint Source Pollution Control Program, 1995.

Coastal Nonpoint Pollution Control Program; Program Development and Approval Guidance, Working Group, Working Paper; USEPA, April 1992.

Manual of Standards of Erosion and Sediment Control Measures, Association of Bay Area Governments, May 1995.

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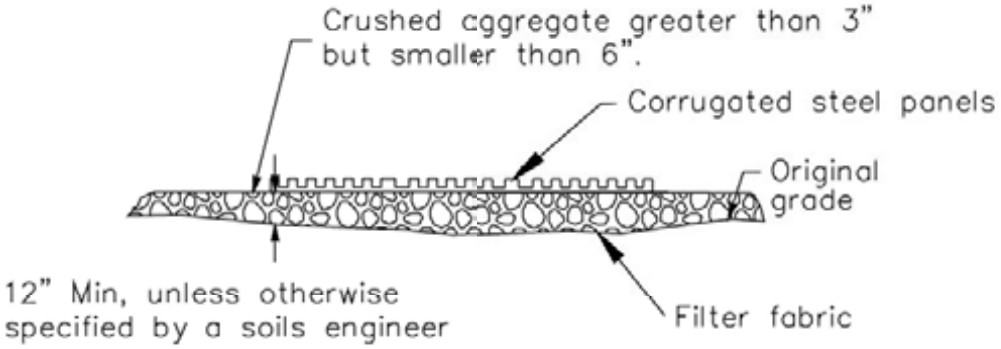
Entrance/Outlet Tire Wash TC-3



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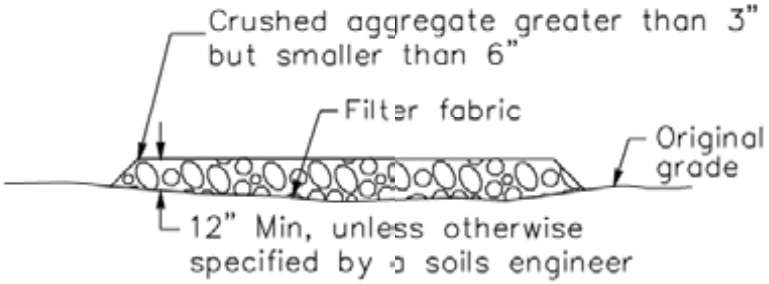

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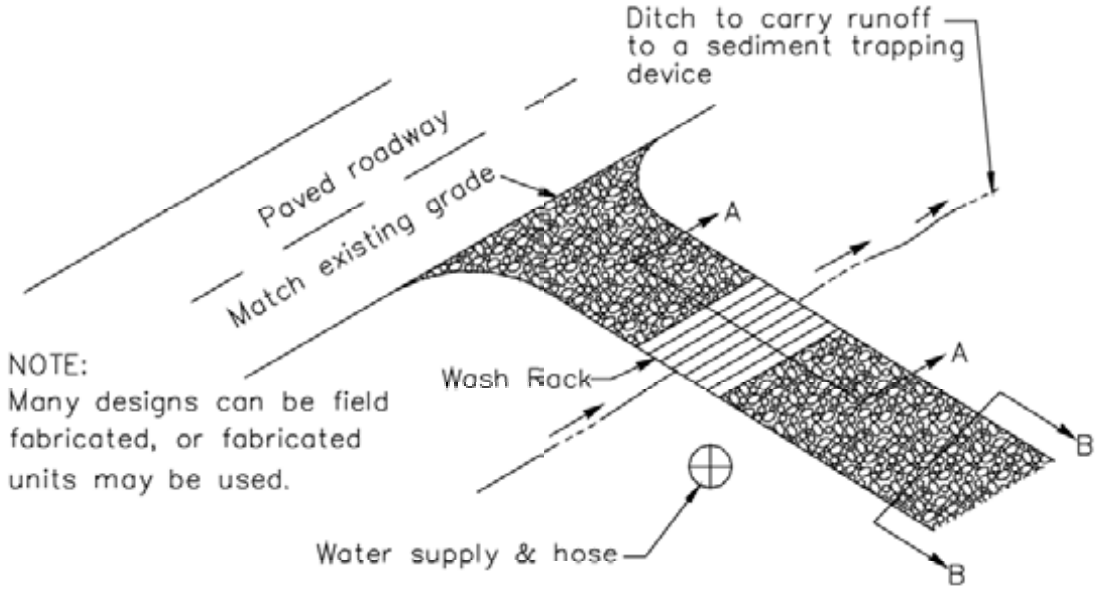
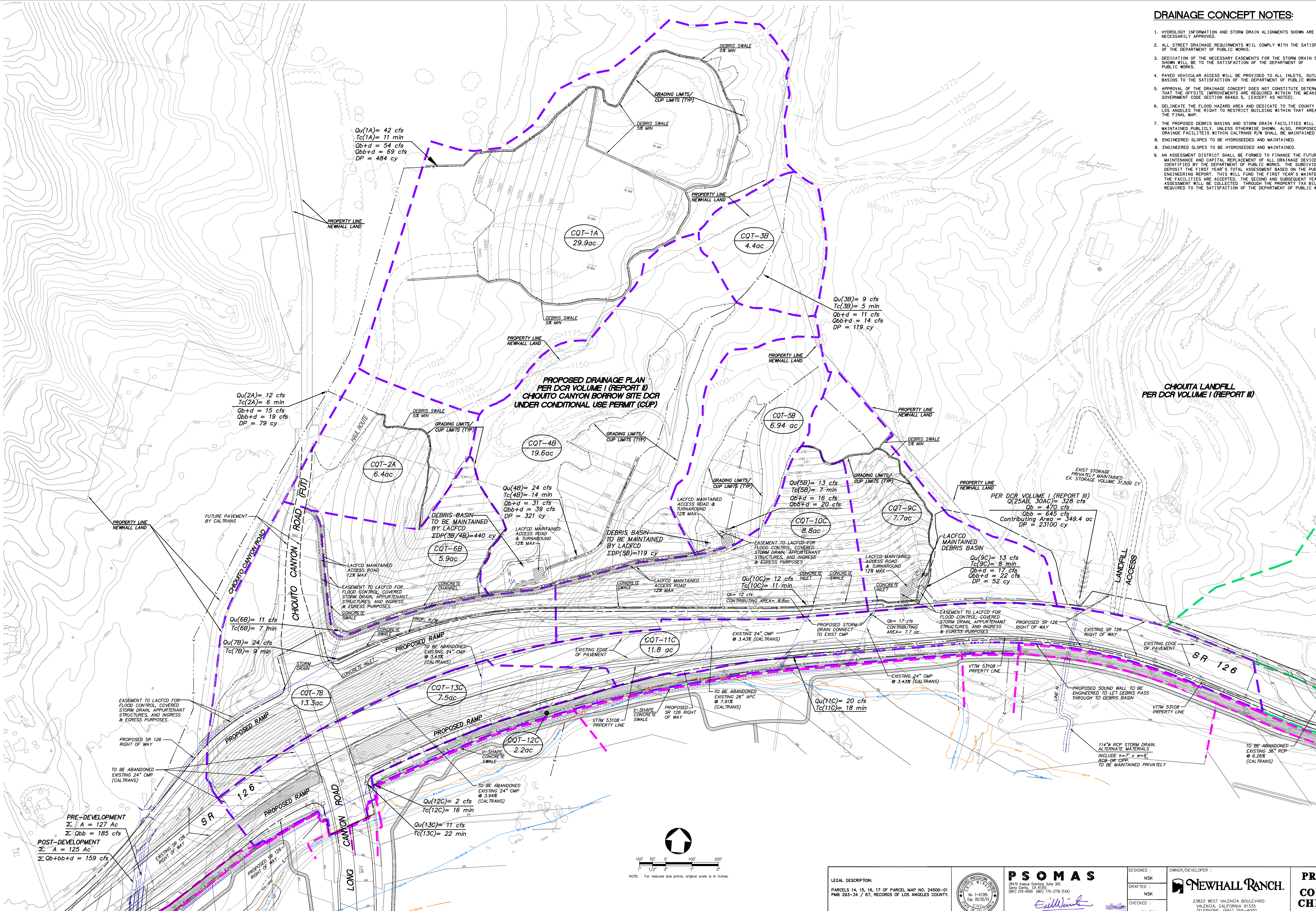


TYPICAL TIRE WASH
NOT TO SCALE

APPENDIX 7

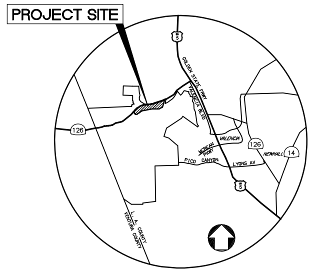
HYDROLOGY EXHIBITS
CHIQUITO CANYON BORROW SITE



DRAINAGE CONCEPT NOTES:

1. HYDROLOGY INFORMATION AND STORM DRAIN ALIGNMENTS SHOWN ARE NOT NECESSARILY APPROVED.
2. ALL STREET DRAINAGE REQUIRMENTS WIIL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
5. APPROVAL OF THE DRAINAGE CONCEPT DOES NOT CONSTITUTE DETERMINATION THAT THE OFFSITE IMPROVEMENTS ARE REQUIRED WITHIN THE MEANING OF GOVERNMENT CODE SECTION 66462.5, (EXCEPT AS NOTED).
6. DELINEATE THE FLOOD HAZARD AREA AND DEDICATE TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT BUILDING WITHIN THAT AREA ON THE FINAL MAP.
7. THE PROPOSED DEBRIS BASINS AND STORM DRAIN FACILITIES WILL BE MAINTAINED PUBLICLY. UNLESS OTHERWISE SHOWN. ALSO, PROPOSED DRAINGE FACILITEIS WITHIN CALTRANS R/W SHALL BE MAINTAINED BY CALTRANS
8. ENGINEERED SLOPES TO BE HYDROSEEDED AND MAINTAINED.
8. ENGINEERED SLOPES TO BE HYDROSEEDED AND MAINTAINED.
9. AN ASSESSMENT DISTRICT SHALL BE FORMED TO FINANCE THE FUTURE ONGOING MAINTENANCE AND CAPITAL REPLACEMENT OF ALL DRAINAGE DEVICES/SYSTEMS IDENTIFIED BY THE DEPARTMENT OF PUBLIC WORKS. THE SUBDIVIDER SHALL DEPOSIT THE FIRST YEAR'S TOTAL ASSESSMENT BASED ON THE PUBLIC WORKS ENGINEERING REPORT. THIS WILL FUND THE FIRST YEAR'S MAINTENANCE AFTER THE FACILITIES ARE ACCEPTED. THE SECOND AND SUBSEQUENT YEARS' ASSESSMENT WILL BE COLLECTED THROUGH THE PROPERTY TAX BILL. THIS IS REQUIRED TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.

PROJECT SITE



VICINITY MAP

NOT TO SCALE

LEGEND:

CQT-5A — SUB-BASIN NUMBER
4.4 ac — SUB-BASIN AREA (ac)

- EXISTING LOT LINE
- EXISTING EASEMENT
- EXISTING CONTOUR
- DAYLIGHT LINE
- SUB-BASIN BOUNDARY
- DRAINAGE AREA BOUNDARY
- RIGHT OF WAY/PROPERTY LINE
- EASEMENT
- PROPOSED CONTOUR
- PROPOSED STORM DRAIN
- EXISTING STORM DRAIN CULVERT
- DEBRIS CHANNEL (5% MIN)
- EXISTING FLOWPATH
- PROPOSED DEBRIS CHANNEL FLOWPATH

(8.6 ac) — UNDEVELOPED PORTION OF SUBAREA

Qu(sub)= 9 cfs — PROPOSED SUBAREA Q (unburned)
Tc(sub)= 15 min — PROPOSED TIME OF CONCENTRATION

PAVED ACCESS ROAD-24FT WIDE

Qu	=	UNDEVELOPED Q
Qd	=	DEVELOPED Q
Qb	=	BURNED Q
Qbb	=	BURNED+BULKED Q
Qb+d	=	BURNED & DEVELOPED Q
Qbb+d	=	BURNED & BULKED & DEVELOPED Q
DP	=	DEBRIS PRODUCTION
Qcap	=	FULL FLOW CAPACITY
SCE	=	SOUTHERN CALIFORNIA EDISON

NOTES:

A BURNED FLOW FOR SUBAREAS CQT-3B, CQT-4B, CQT-6B, CQT-5B, CQT-9C BURNED AND BULKED FLOW FOR SUBAREAS CQT-1A and, CQT-2A, DEVELOP FLOW FOR SUBAREAS CQT-7B, CQT-10C, CQT-11C , CQT-12C AND CQT-13C

B DRAINAGE TRIBUTARY AREA IS LESS THAN IN EXISTING CONDITION BECAUSE THE AREA SOUTH OF SR-126 IS PART OF LANDMARK VILLAGE DEVELOPMENT TRACT 53108 IN THE PROPOSED CONDITION.

C AREAS CQT-1A AND CQT-2A DRAIN DIRECTLY TO CHIQUITO CANYON. EXISTING CONDITION IS MAINTAINED.

K = Burn Factor from Appendix G-2, $K = 0.677*(1^{-0.102})$

$Qb = A*I*(1-K)+K*Qu$

$Qbb = 1.27*Qb$

$DP = A*27$ cy (DPA Zone 9)

DRAINAGE CONCEPT/STORM WATER QUALITY PLAN
IS CONCEPTUALLY APPROVED
LAND DEVELOPMENT DIVISION
LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

PROJECT AREA

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

LEGAL DESCRIPTION:

PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER — ERIK WINATA — No. C-67285 — Exp. 09/30/10 — CIVIL — STATE OF CALIFORNIA

PSOMAS
28470 Avenue Stanford, Suite 300
Santa Clarita, CA 91355
(661) 219-6000 (661) 775-2718 (FAX)

ERIK WINATA R.C.E. NO. C-67285 DATE

DESIGNED : NSK
DRAFTED : NSK
CHECKED : EW/RB

OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

PROPOSED DRAINAGE PLAN FOR CONDITIONAL USE PERMIT (CUP) CHIQUITO CANYON BORROW SITE

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08

SHEET 1 OF 2

RV067-01

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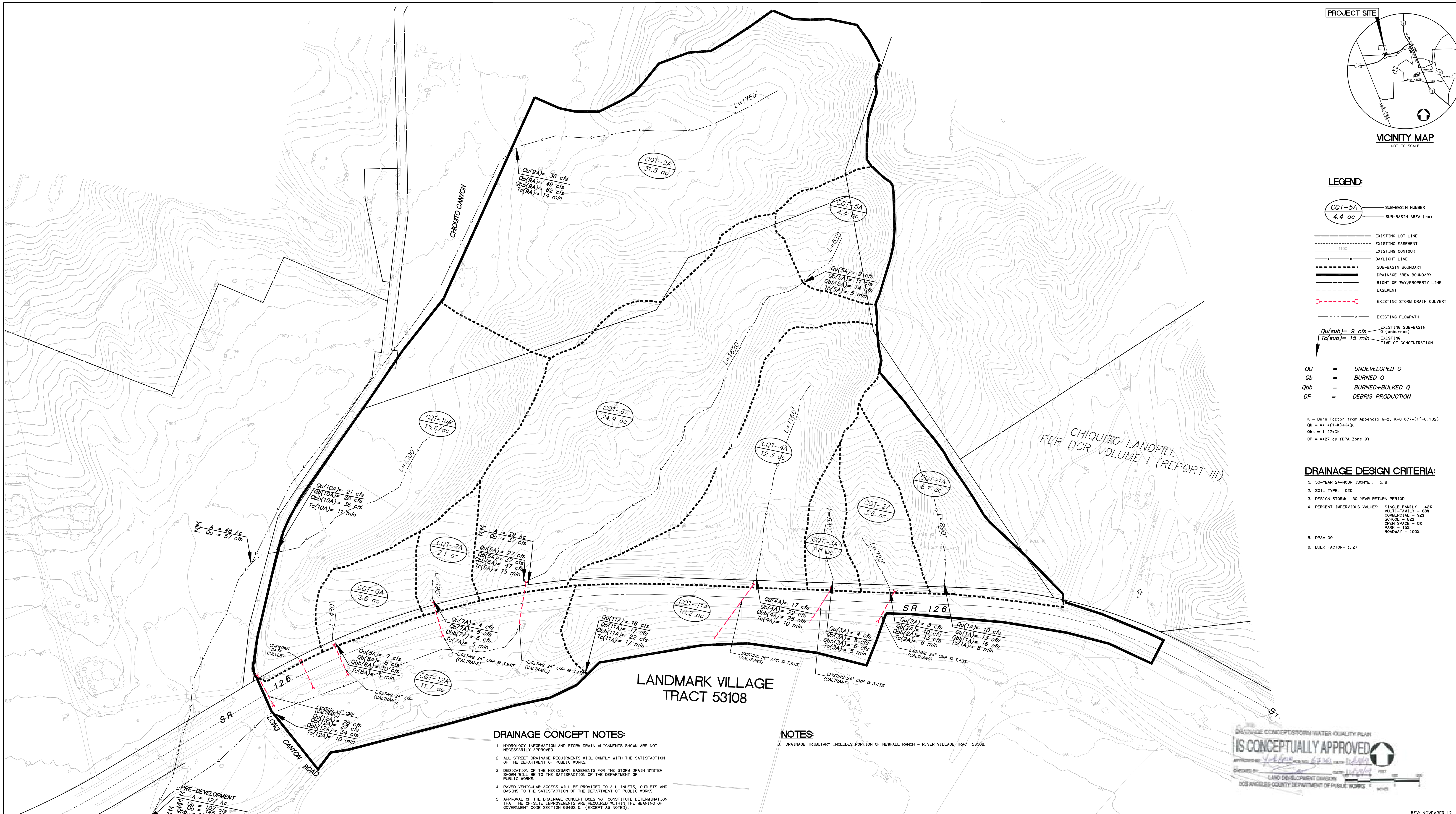

QU = UNDEVELOPED Q
Qb = BURNED Q
Qbb = BURNED+BULKED Q
DP = DEBRIS PRODUCTION

K = Burn Factor from Appendix G-2, K=0.677*(1^-0.102)
Qb = A*I*(1-K)+K*Qu
Qbb = 1.27*Qb
DP = A*27 cy (DPA Zone 9)

DRAINAGE DESIGN CRITERIA:

1. 50-YEAR 24-HOUR ISOHYET: 5.8
2. SOIL TYPE: 020
3. DESIGN STORM: 50 YEAR RETURN PERIOD
4. PERCENT IMPERVIOUS VALUES: SINGLE FAMILY - 42%, MULTI-FAMILY - 68%, COMMERCIAL - 92%, SCHOOL - 82%, OPEN SPACE - 0%, PARK - 15%, ROADWAY - 100%
5. DPA= 09
6. BULK FACTOR= 1.27

DRAINAGE CONCEPT NOTES:

1. HYDROLOGY INFORMATION AND STORM DRAIN ALIGNMENTS SHOWN ARE NOT NECESSARILY APPROVED.
2. ALL STREET DRAINAGE REQUIRMENTS WIIL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
5. APPROVAL OF THE DRAINAGE CONCEPT DOES NOT CONSTITUTE DETERMINATION THAT THE OFFSITE IMPROVEMENTS ARE REQUIRED WITHIN THE MEANING OF GOVERNMENT CODE SECTION 66462.5, (EXCEPT AS NOTED).
6. DELINEATE THE FLOOD HAZARD AREA AND DEDICATE TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT BUILDING WITHIN THAT AREA ON THE FINAL MAP.

NOTES:

A DRAINAGE TRIBUTARY INCLUDES PORTION OF NEWHALL RANCH - RIVER VILLAGE TRACT 53108.

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

LEGAL DESCRIPTION:

PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

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ERIK WINATA R.C.E. NO. C-67285 DATE

DESIGNED : JY/SW/EW
DRAFTED : AC/ML
CHECKED : MG/RB

OWNER/DEVELOPER :
NEWHALL RANCH
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

EXISTING DRAINAGE PLAN FOR CONDITIONAL USE PERMIT (CUP) CHIQUITO CANYON BORROW SITE

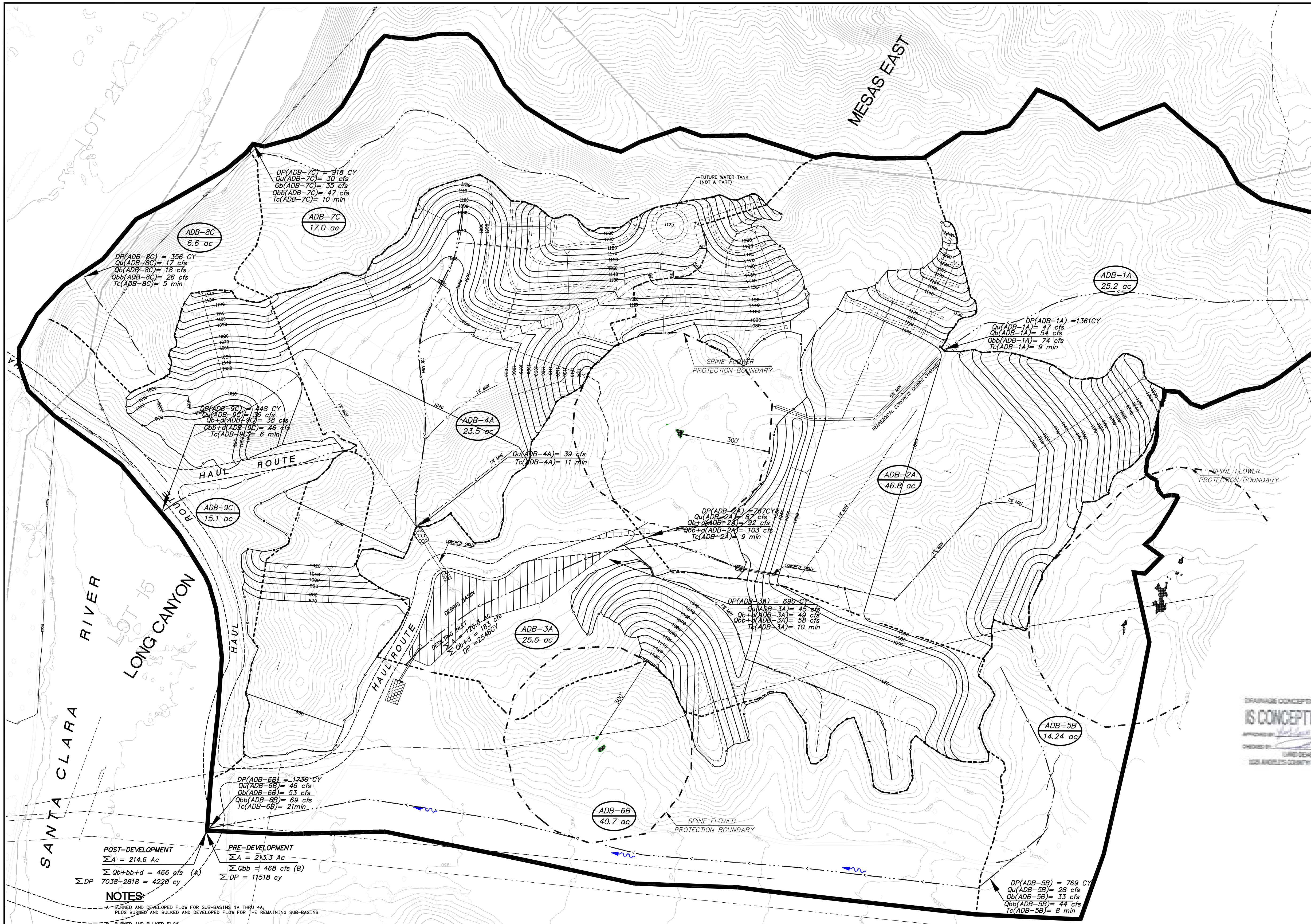
IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 2 OF 2

RV067-02

APPENDIX 8

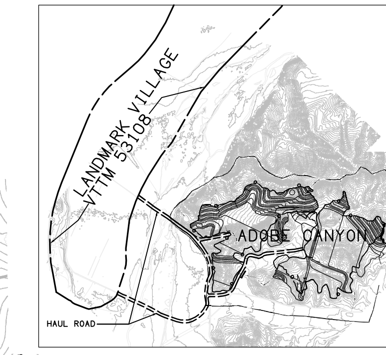
HYDROLOGY EXHIBITS
ADOBE CANYON BORROW SITE



NOTES:

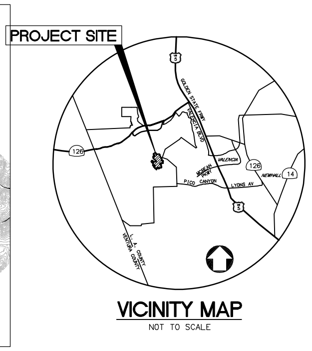
A BURNED AND DEVELOPED FLOW FOR SUB-BASINS 1A THRU 4A; PLUS BURNED AND BULKED AND DEVELOPED FLOW FOR THE REMAINING SUB-BASINS.

B BURNED AND BULKED FLOW



KEY MAP : ADOBE CANYON SITE

FEET 1000 500 0 1000 2000
INCHES 1 1/2 0 1 2



VICINITY MAP

NOT TO SCALE

LEGEND:

- ADB-1A — SUB-BASIN NUMBER
- 26.0 ac — SUB-BASIN AREA (ac)
- EXISTING PROPERTY LINE
- EXISTING EASEMENT
- EXISTING CONTOUR (1100)
- DAYLIGHT LINE
- CUT & FILL
- FLOW LINE
- BENCH
- CONCRETE DEBRIS SWALE
- HAUL ROUTE ACCESS ROAD
- RIVER CORRIDOR-2003 (FORMERLY SEA 23) (RC03)
- FEMA
- Q-CAP (Q50)
- DEBRIS TRIBUTARY LIMIT
- SUB-BASIN BOUNDARY
- DRAINAGE AREA BOUNDARY
- RIGHT OF WAY
- EASEMENT
- PROPOSED CONTOUR (950)
- DAYLIGHT LINE
- STORM DRAIN (SD)
- EXISTING CMP INLET
- FLOWPATH
- DEBRIS FLOWPATH
- DEBRIS BASIN
- SPINE FLOWER
- SPINE FLOWER PROTECTION BOUNDARY
- COAST LIVE OAK (Quercus agrifolia)
- VALLEY OAK (Quercus lobata)
- SCRUB OAK (Quercus berberidifolia)
- SOUTHERN CALIFORNIA BLACK WALNUT (Juglans california)
- HOLLYLEAF CHERRY (Prunus ilicifolia)
- EXISTING TREE CANOPY
- (8.6 ac) — UNDEVELOPED PORTION OF SUBAREA
- Qu(sub)= 9 cfs — PROPOSED SUBAREA Q(unburned)
- Tc(sub)= 15 min — PROPOSED TIME OF CONCENTRATION

Qu	=	UNDEVELOPED Q
Qd	=	DEVELOPED Q
Qb	=	BURNED Q
Qbb	=	BURNED+BULKED Q
Qb+d	=	BURNED & DEVELOPED Q
Qbb+d	=	BURNED & BULKED & DEVELOPED Q
DP	=	DEBRIS PRODUCTION

(1) – K = Burn Factor from Appendix G-2, K = 0.677*(I ^-0.102)

(2) – Qb = A*I*(1-K)+K*Qu

(3) – Qbb =1.36*Qb

(4) – DP = A*54 cy (Zone 8)

DRAINAGE CONCEPT NOTES:

1. HYDROLOGY INFORMATION AND STORM DRAIN ALIGNMENTS SHOWN ARE NOT NECESSARILY APPROVED.
2. ALL STREET DRAINAGE REQUIRMENTS WIIL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS, AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
5. APPROVAL OF THE DRAINAGE CONCEPT DOES NOT CONSTITUTE DETERMINATION THAT THE OFFSITE IMPROVEMENTS ARE REQUIRED WITHIN THE MEANING OF GOVERNMENT CODE SECTION 66462.5, (EXCEPT AS NOTED).
6. DELINEATE THE FLOOD HAZARD AREA AND DEDICATE TO THE COUNTY OF LOS ANGELES THE RIGHT TO RESTRICT BUILDING WITHIN THAT AREA ON THE FINAL MAP.
7. THE PROPOSED DEBRIS BASINS AND STORM DRAIN FACILITIES WILL BE PRIVATELY MAINTAINED.
8. ENGINEERED SLOPES TO BE HYDROSEEDED AND MAINTAINED.
9. DESILTING INLETS, CONC. SWALES AND OTHER DRAINAGE DEVICE TO BE PRIVATELY MAINTAINED

FEET 100 50 0 100 200
INCHES 1 1/2 0 1 2

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

NEWHALL RANCH.

LEGAL DESCRIPTION:

PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

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ERIK WINATA R.C.E. NO. C-67285 DATE

DESIGNED : EW/NSK
DRAFTED : AC/ML
CHECKED : MG/RB

OWNER/DEVELOPER :
NEWHALL RANCH.
23823 WEST VALENCIA BOULEVARD
VALENCIA, CALIFORNIA 91335
TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

DRAINAGE CONCEPT PLAN FOR CONDITIONAL USE PERMIT (CUP) ADOBE CANYON BORROW SITE

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08

SHEET 1 OF 2

RV068-01

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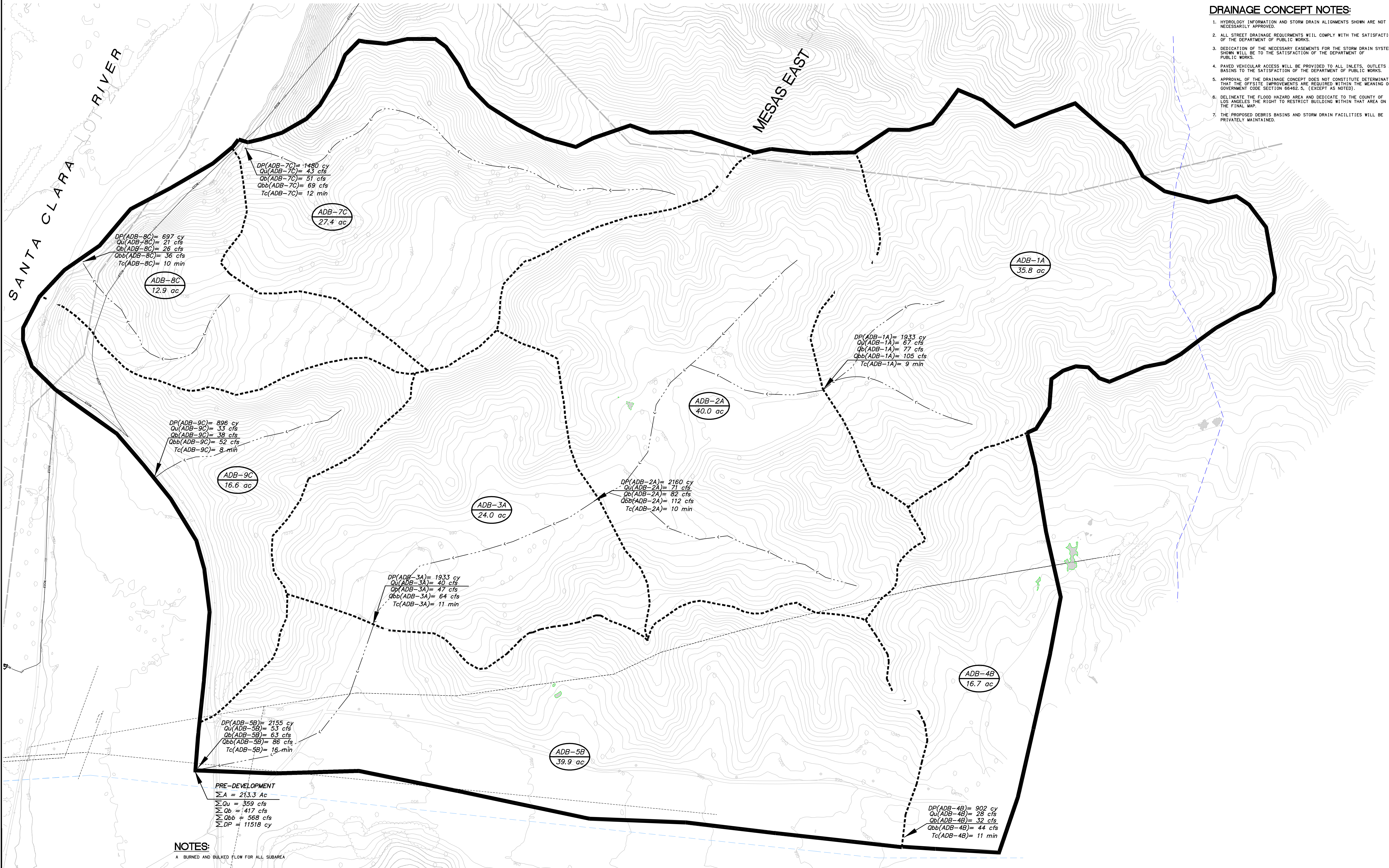

DRAINAGE CONCEPT NOTES:

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2. ALL STREET DRAINAGE REQUIRMENTS WIIL COMPLY WITH THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
3. DEDICATION OF THE NECESSARY EASEMENTS FOR THE STORM DRAIN SYSTEM SHOWN WILL BE TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
4. PAVED VEHICULAR ACCESS WILL BE PROVIDED TO ALL INLETS, OUTLETS AND BASINS TO THE SATISFACTION OF THE DEPARTMENT OF PUBLIC WORKS.
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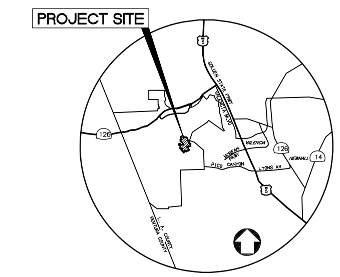

VICINITY MAP
NOT TO SCALE

LEGEND:

- ADB-1A / 35.8 ac — SUB-BASIN NUMBER; SUB-BASIN AREA (ac)
- SUB-BASIN BOUNDARY
- DRAINAGE AREA BOUNDARY
- RIGHT OF WAY
- EASEMENT
- 950 — PROPOSED CONTOUR
- DAYLIGHT LINE
- SD — STORM DRAIN
- EXISTING CMP INLET
- FLOWPATH
- DEBRIS FLOWPATH
- RIVER CORRIDOR-2003 (FORMERLY SEA 23)
- FEMA
- Q-CAP
- DEBRIS TRIBUTARY LIMIT
- SPINE FLOWER
- COAST LIVE OAK (Quercus agrifolia)
- VALLEY OAK (Quercus lobata)
- SCRUB OAK (Quercus berberidifolia)
- SOUTHERN CALIFORNIA BLACK WALNUT (Juglans californica)
- HOLLYLEAF CHERRY (Prunus ilicifolia)
- EXISTING TREE CANOPY
- Qu(sub)= 9 cfs — EXISTING SUB-BASIN Q(unburned)
- Tc(sub)= 15 min — EXISTING TIME OF CONCENTRATION

QU = UNDEVELOPED Q
Qb = BURNED Q
Qbb = BURNED+BULKED Q
DP = DEBRIS PRODUCTION

K = Burn Factor from Appendix G-2, K=0.677*(1^-0.102)
Qb = A*I*(1-K)+K*Qu
Qbb = 1.36*Qb
DP = A*54 cy (DPA Zone 8)

DRAINAGE CONCEPT/STORM WATER QUALITY PLAN IS CONCEPTUALLY APPROVED
LAND DEVELOPMENT DIVISION
LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

FEET 100 50 0 100 200
INCHES 1 1/2 0 1 2

NOTES:

A BURNED AND BULKED FLOW FOR ALL SUBAREA

NEWHALL RANCH

PROJECT AREA

REV: NOVEMBER 12, 2009
DATE: SEPT 15, 2004

LEGAL DESCRIPTION:
PARCELS 14, 15, 16, 17 OF PARCEL MAP NO. 24500-01 PMB 293-34 / 67, RECORDS OF LOS ANGELES COUNTY.

REGISTERED PROFESSIONAL ENGINEER ERIK WINATA No. C-67285 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

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DESIGNED : JY/SW/EW
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CHECKED : MG/RB

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TELEPHONE: (661) 255-4000
REPRESENTATIVE : MR. FRED MACMURDO

EXISTING DRAINAGE PLAN FOR CONDITIONAL USE PERMIT (CUP) ADOBE CANYON BORROW SITE

IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA

PROJECT NO. 1NRC0107.08
SHEET 2 OF 2

RV068-02

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REPORT III

OFF-SITE CHIQUITO LANDFILL DRAINAGE CONCEPT REPORT

NOTE

Off-site Chiquita Landfill Drainage Concept Report was conceptually approved by Land Development Division, Los Angeles County Department of Public Works on 07/05/07. There are no changes made since the approval of this report.

PSOMAS

DRAINAGE CONCEPT/STORM WATER QUALITY PLAN
IS CONCEPTUALLY APPROVED
APPROVED BY: [signature] RCE NO. 67363 DATE: 11/29/06
CHECKED BY: [signature] DATE: 11/29/06
LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
LAND DEVELOPMENT DIVISION

OFF-SITE CHIQUITO LANDFILL DRAINAGE CONCEPT

Psomas Project No: 1NRCO10708

November 17, 2006

Prepared for:

RECEIVED
NOV 20
PROCESSING CENTER
LAND DEVELOPMENT DIVISION

NEWHALL LAND
23823 W. Valencia Blvd.
Valencia, CA 91355-2194
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Prepared by:

PSOMAS
28470 Avenue Stanford, Suite 300
Santa Clarita, California 91355
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REGISTERED PROFESSIONAL ENGINEER
ERIK WINATA
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STATE OF CALIFORNIA
11/12/09

REGISTERED PROFESSIONAL ENGINEER
ROSS WILLIAM BARKER
No. C 32799
Exp. 6/30/08
CIVIL
STATE OF CALIFORNIA

11/17/2006

Ross W. Barker, P.E. R.C.E. No. C32799 Exp. Date 06-30-08

TABLE OF CONTENTS

SECTION	PAGE
Introduction	1
Project Background	
Purpose and Scope	
Hydrology	2
General Approach	
Data Sources	
Watershed Characteristics	
Existing Condition Results	
Standard Urban Stormwater Mitigation Plan (SUSMP)	3
Limitations	4
References	5

APPENDICES

A. **Existing Condition TC Calculations**

B. **Existing Condition LACDPW MORA Calculations (Clear)**

C. **Existing Condition LACDPW MORA Calculations (burned)**

D. **Hydrology/Sedimentation (LAC) Manual P-2 and P-5**

E. **Hydrologic/Drainage Map**

1.0 Introduction

Project Background

Psomas has been retained by the Newhall Land Co. to prepare a drainage concept report for the landfill area associated with the Tentative Tract Map No. 53108 for the proposed Landmark Village development. The landfill area is under a Conditional Use Permit (CUP), not part of the tentative tract map itself. The Chiquito landfill site is located north of Landmark Village, just north of SR 126.

Purpose and Scope

The project falls under the jurisdiction of the Los Angeles County Department of Public Works (LACDPW). The purpose of this drainage concept report is:

- To meet Los Angeles County Land Development requirements in support of the tentative tract map submittal, allowing final design and construction to proceed in a timely manner;
- To determine the runoff for existing hydrologic conditions;
- To provide sufficient detailed information to support debris basin facilities,

It should be noted that detailed storm drain and debris basin analyses is beyond the scope of this drainage concept.

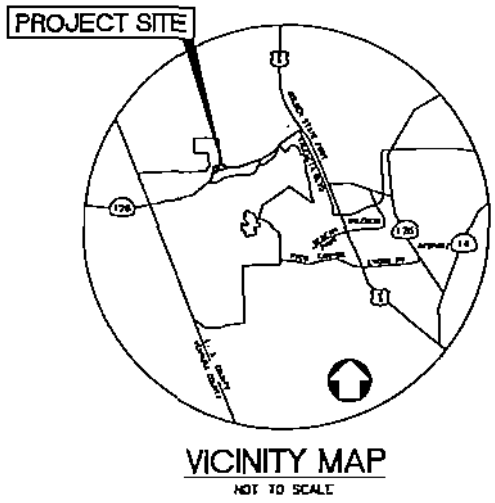


Figure 1. Location Map

2.0 Hydrology

General Approach

The watershed for the landfill area was divided into several sub-basins that were identified and characterized for existing conditions. Computer modeling was used to estimate the runoff for the 50-year storm events. The sub-basins drain southwesterly on site, ultimately tributary to the Santa Clara River. The analysis considered with the additional bulking factors where appropriate to size sediment/debris facilities.

Data Sources

The primary sources of data were the *LACDPW Hydrology/ Sedimentation Manual and Appendices* (LACDPW 1991, 1992, 1993, 2002) and the Los Angeles County *Standard Urban Stormwater Mitigation Plan* (March 2000). Other key sources included the *TC (v1.0),* and *MORA (v7.2)* manuals and the Newhall Ranch Specific Plan Master Hydrology and Drainage Concept, Sikand, December 1999 (appended for reference).

Watershed Characteristics

The Chiquito landfill site is bounded by SR 126 on the south, Chiquito borrow site on the west, and the existing Post Office site and Parcel Map 18108 on the east. The total drainage area for the site encompasses approximately 349 acres and flows predominantly south toward SR 126. The majority of the area is undeveloped land with steep to moderate slopes.

The soil of the landfill site is classified by Los Angeles County to be Type 020. The drainage area of the site is associated with Debris Potential Area (DPA) number 05. The peak bulking factor curves and peak debris production rates used in the analysis are included in Appendix D (LACDPW P2 & P5).

The Chiquito Landfill site is located in Los Angeles County's Rainfall Isohyets 5.8.

The *LACDPW TC (TC_calc_depth.xls, June2002)* program was used to calculate the time of concentration for each sub area. The *LACDPW MORA (v7.2)* computer program was used for the *Modified Rational Method* calculations and calculation of peak runoff rates and hydrographs. The TC calculations and MORA program output for clear and burned flow condition are provided in Appendix 1, 2 and 3, respectively. In accordance with LACDPW requirements, the 50-year burned; bulked storm event was used as the main design storm in the analysis.

Existing Condition Results

Existing condition hydrology results for the Capital Flood storm event are summarized in Tables 1 and 2 below.

Table 1
Existing Condition Hydrology Summary – Chiquito Landfill Site

EXISTING CONDITION						
INCREMENTAL SUB-BASINS						
Sub-Basin Number	**Time of Concentration (Tc)**	**Sub-Basin Area (acres)**	**Q50 Unburned (cfs)**	**Q50 Burned (cfs)**	**Q50 Burned Bulked (cfs)**	**Debris Volume (Yd^3)**
1A	11	18.5	25	33	-	-
2A	9	21.6	35	42	-	-
3A	16	37.7	38	54	-	-
4A	16	33.2	34	47	-	-
5A	10	30.9	38	53	-	-
10A	13	13.1	18	21	-	-
12A	16	32.7	34	47	-	-
6B	10	14.8	21	28	-	-
7B	10	9.5	14	18	-	-
13B	12	18.7	23	31	-	-
14B	14	20.9	24	32	-	-
15B	17	28.0	28	40	-	-
18C	14	23.1	28	35	-	-
19C	16	30.4	31	43	-	-
25C	9	10.3	20	40	-	-
TOTAL	---	349.4	328	470	645*	23,100**

** Bulking factor is taken form Hydrology/Sedimentation Appendix P-5*
*** Debris production rate is taken from P-2*

Table 2
Land Use Impervious Values

Land Use	Percent Imperviousness
Open Space/Natural Landfill	0%

3.0 Standard Urban Stormwater Mitigation Plan (SUSMP)

Instead of volume- or flow-based Treatment Control BMPs, the Los Angeles County Standard Urban Stormwater Mitigation Plan (SUSMP) requirements will be met by Erosion and Sediment Control BMPs, which decrease the potential of slopes and channels to erode and impact stormwater runoff. Los Angeles County SUSMP guidelines specify BMPs such as the following:

- Conveying runoff safely from the tops of slopes and stabilizing disturbed slopes
- Utilizing natural drainage systems to the maximum extent practicable (MEP)
- Stabilizing permanent channel crossings
- Vegetating slopes with native or drought tolerant vegetation
- Installing energy dissipators, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion

Final selection and detailed design of these BMPs is beyond the scope of this report. However, the report and exhibits show that erosion and sediment control SUSMP requirements can be met.

4.0 Limitations

This report was prepared to comply with the guidelines established by the County of Los Angeles and their representatives. Evaluation of the appropriateness of these guidelines and the accuracy of the County data were beyond the scope of this work.

Usage of the report is limited to address the purpose and scope previously defined by the project owner, Newhall Land. Psomas shall not be held responsible for any unauthorized application of this report and the contents herein.

The opinions represented in this report have been derived in accordance with current standards of civil engineering practice. No other warranty is expressed or implied.

5.0 References

Los Angeles County Department of Public Works, *LACDPW Hydrology/ Sedimentation Manual and Appendices* (1991, 1992, 1993, 2002)

Los Angeles County Department of Public Works, *LACDPW TC v1.0 Manual, TC_calc_depth.xls* (December 1991, June 2002)

Los Angeles County Department of Public Works, *LACDPW Modified Rational Method, MORA (MORA) v7.2 Manual* (September 2002)

Newhall Ranch, Newhall Ranch Specific Plan Master Hydrology and Drainage Concept (Sikand, December 1999)

APPENDICES

APPENDIX A

TC CALCULATIONS: EXISTING CONDITION

Project	Subarea	Area (acres	%imp				
Landfill	1A	18.5		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	2A	21.6		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	3A	37.7		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	4A	33.2		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	5A	35.9		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	10A	13.1		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	12A	32.7		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	6B	14.8		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	7B	9.5		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	13B	18.7		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	14B	20.9		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	15B	29		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	18C	23.1		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	19C	30.4		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			
Landfill	26C	10.3		7*(Cd*I)^-0.519*(L)^0.483*(S)^-0.135			

APPENDIX B

LACDPW MORA CALCULATIONS: EXISTING CONDITION CLEAR

MODIFIED RATIONAL METHOD HYDROLOGY - STORM YEAR = 50

CHIQUITO LANDFILL HYDROLOGY - EXISTING Condition STORM DAY 4

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	1A	18.5	25.	18.5	25.	2	1060.	.19300	.00	.00	0.	20	11	A29	.00
300	2A	21.6	33.	40.1	56.	2	900.	.05000	.00	.00	0.	20	9	A29	.00
300	3A	37.7	39.	77.8	92.	2	440.	.06800	.00	.00	0.	20	16	A29	.00
300	4A	33.2	34.	111.0	125.	0	0.	.00000	.00	.00	0.	20	16	A29	.00
300	5A	35.9	39.	146.9	162.	0	0.	.00000	.00	.00	0.	20	15	A29	.00
300	6B	14.8	21.	14.8	21.	2	940.	.11200	.00	.00	0.	20	10	A29	.00
300	7B	9.5	14.	24.3	33.	0	0.	.00000	.00	.00	0.	20	10	A29	.00

CONFLUENCE Q'S

300 8A TA 1158 QA 162. QAB 194. QB 32. 300 8B TB 1156 QB 33. QBA 192. QA 159.
300 8AB TAB 1157 QAB 195. QA 162. QB 33.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	8AB	24.3	33.	171.2	195.	0	0.	.00000	.00	.00	0.	20	0	A29	.00
300	9A	.0	0.	171.2	195.	2	1240.	.03600	.00	.00	0.	20	99	A29	.00
300	10A	13.1	16.	184.3	205.	2	1470.	.00800	.00	.00	0.	20	13	A29	.00
300	11A	.0	0.	184.3	184.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	12A	32.7	34.	217.0	205.	0	0.	.00000	.00	.00	0.	20	16	A29	.00
300	13B	18.7	23.	18.7	23.	2	1130.	.02700	.00	.00	0.	20	12	A29	.00
300	14B	20.9	24.	39.6	43.	2	1940.	.07500	.00	.00	0.	20	14	A29	.00
300	15B	29.0	29.	68.6	66.	2	860.	.07200	.00	.00	0.	20	17	A29	.00
300	16B	.0	0.	68.6	65.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	17B	.0	0.	68.6	65.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	18C	23.1	26.	23.1	26.	2	1480.	.12200	.00	.00	0.	20	14	A29	.00
300	19C	30.4	31.	53.5	56.	0	0.	.00000	.00	.00	0.	20	16	A29	.00

CONFLUENCE Q'S

300 20B TB 1164 QB 65. QBC 112. QC 46. 300 20C TC 1158 QC 56. QCB 110. QB 55.
300 20BC TBC 1161 QBC 116. QB 63. QC 53.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	20BC	53.5	56.	122.1	116.	0	0.	.00000	.00	.00	0.	20	0	A29	.00
300	21B	.0	0.	122.1	116.	2	570.	.00400	.00	.00	0.	20	99	A29	.00
300	22B	.0	0.	122.1	113.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	23B	.0	0.	122.1	113.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	24B	.0	0.	122.1	113.	0	0.	.00000	.00	.00	0.	20	99	A29	.00

CONFLUENCE Q'S

300 25A TA 1165 QA 205. QAB 318. QB 113. 300 25B TB 1165 QB 113. QBA 318. QA 205.
300 25AB TAB 1165 QAB 318. QA 205. QB 113.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	25AB	122.1	113.	339.1	318.	0	0.	.00000	.00	.00	0.	20	0	A29	.00

Program Package Serial Number: 2033

MODIFIED RATIONAL METHOD HYDROLOGY - STORM YEAR = 50

CHIQUITO LANDFILL HYDROLOGY - EXISTING Condition STORM DAY 4

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	26C	10.3	16.	10.3	16.	2	1080.	.01600	.00	.00	0.	20	9	A29	.00
300	27A	.0	0.	339.1	318.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	28A	.0	0.	339.1	318.	0	0.	.00000	.00	.00	0.	20	99	A29	.00
300	29A	.0	0.	339.1	318.	0	0.	.00000	.00	.00	0.	20	99	A29	.00

CONFLUENCE Q'S

300 30A TA 1165 QA 318. QAC 328. QC 10. 300 30C TC 1161 QC 13. QCA 285. QA 272.
300 30AC TAC 1165 QAC 328. QA 318. QC 10.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	30AC	10.3	13.	349.4	328.	0	0.	.00000	.00	.00	0.	20	0	A29	.00
300	31A	.0	0.	349.4	328.	0	0.	.00000	.00	.00	0.	20	99	A29	.00

APPENDIX C

LACDPW MORA CALCULATIONS: BURNED CONDITION

MODIFIED RATIONAL METHOD HYDROLOGY - STORM YEAR = 50

CHIQUITO LANDFILL HYDROLOGY - EXISTING Condition STORM DAY 4

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	1A	18.5	33.	18.5	33.	2	1060.	.19300	.00	.00	0.	220	11	A29	.00
300	2A	21.6	42.	40.1	73.	2	900.	.05000	.00	.00	0.	220	9	A29	.00
300	3A	37.7	54.	77.8	125.	2	440.	.06800	.00	.00	0.	220	16	A29	.00
300	4A	33.2	47.	111.0	170.	0	0.	.00000	.00	.00	0.	220	16	A29	.00
300	5A	35.9	53.	146.9	222.	0	0.	.00000	.00	.00	0.	220	15	A29	.00
300	6B	14.8	28.	14.8	28.	2	940.	.11200	.00	.00	0.	220	10	A29	.00
300	7B	9.5	18.	24.3	44.	0	0.	.00000	.00	.00	0.	220	10	A29	.00

CONFLUENCE Q'S

300 8A TA 1157 QA 222. QAB 265. QB 43. 300 8B TB 1156 QB 44. QBA 263. QA 219.
300 8AB TAB 1157 QAB 265. QA 222. QB 43.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	8AB	24.3	44.	171.2	265.	0	0.	.00000	.00	.00	0.	220	0	A29	.00
300	9A	.0	0.	171.2	265.	2	1240.	.03600	.00	.00	0.	220	99	A29	.00
300	10A	13.1	21.	184.3	281.	2	1470.	.00800	.00	.00	0.	220	13	A29	.00
300	11A	.0	0.	184.3	260.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	12A	32.7	47.	217.0	294.	0	0.	.00000	.00	.00	0.	220	16	A29	.00
300	13B	18.7	31.	18.7	31.	2	1130.	.02700	.00	.00	0.	220	12	A29	.00
300	14B	20.9	32.	39.6	59.	2	1940.	.07500	.00	.00	0.	220	14	A29	.00
300	15B	29.0	40.	68.6	93.	2	860.	.07200	.00	.00	0.	220	17	A29	.00
300	16B	.0	0.	68.6	92.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	17B	.0	0.	68.6	92.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	18C	23.1	35.	23.1	35.	2	1480.	.12200	.00	.00	0.	220	14	A29	.00
300	19C	30.4	43.	53.5	77.	0	0.	.00000	.00	.00	0.	220	16	A29	.00

CONFLUENCE Q'S

300 20B TB 1163 QB 92. QBC 160. QC 68. 300 20C TC 1157 QC 77. QCB 154. QB 77.
300 20BC TBC 1161 QBC 163. QB 90. QC 73.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	20BC	53.5	77.	122.1	163.	0	0.	.00000	.00	.00	0.	220	0	A29	.00
300	21B	.0	0.	122.1	163.	2	570.	.00400	.00	.00	0.	220	99	A29	.00
300	22B	.0	0.	122.1	160.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	23B	.0	0.	122.1	160.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	24B	.0	0.	122.1	160.	0	0.	.00000	.00	.00	0.	220	99	A29	.00

CONFLUENCE Q'S

300 25A TA 1164 QA 294. QAB 455. QB 160. 300 25B TB 1165 QB 160. QBA 452. QA 292.
300 25AB TAB 1164 QAB 455. QA 294. QB 160.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	25AB	122.1	160.	339.1	455.	0	0.	.00000	.00	.00	0.	220	0	A29	.00

MODIFIED RATIONAL METHOD HYDROLOGY - STORM YEAR = 50

CHIQUITO LANDFILL HYDROLOGY - EXISTING Condition STORM DAY 4

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	26C	10.3	20.	10.3	20.	2	1080.	.01600	.00	.00	0.	220	9	A29	.00
300	27A	.0	0.	339.1	455.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	28A	.0	0.	339.1	455.	0	0.	.00000	.00	.00	0.	220	99	A29	.00
300	29A	.0	0.	339.1	455.	0	0.	.00000	.00	.00	0.	220	99	A29	.00

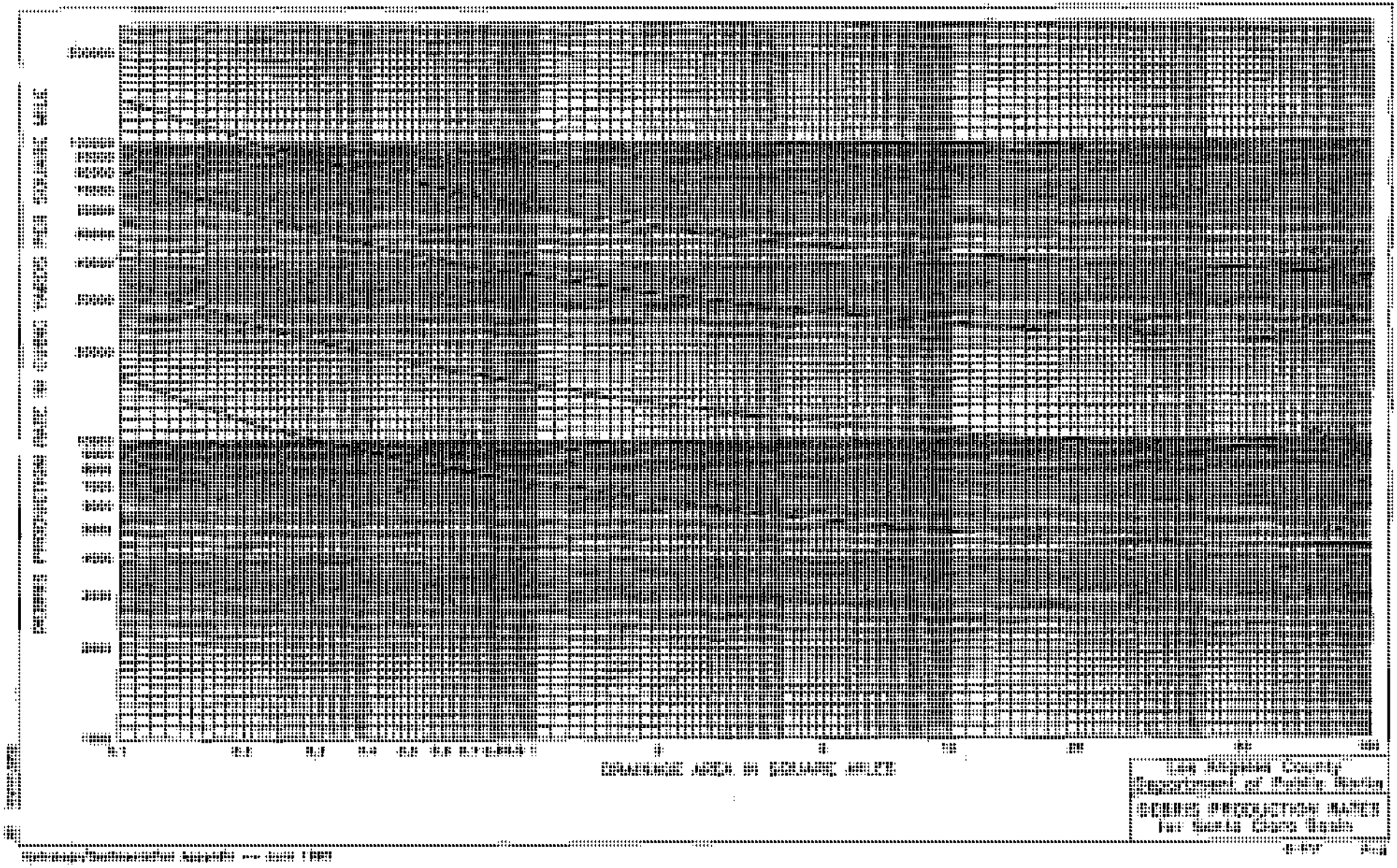
CONFLUENCE Q'S

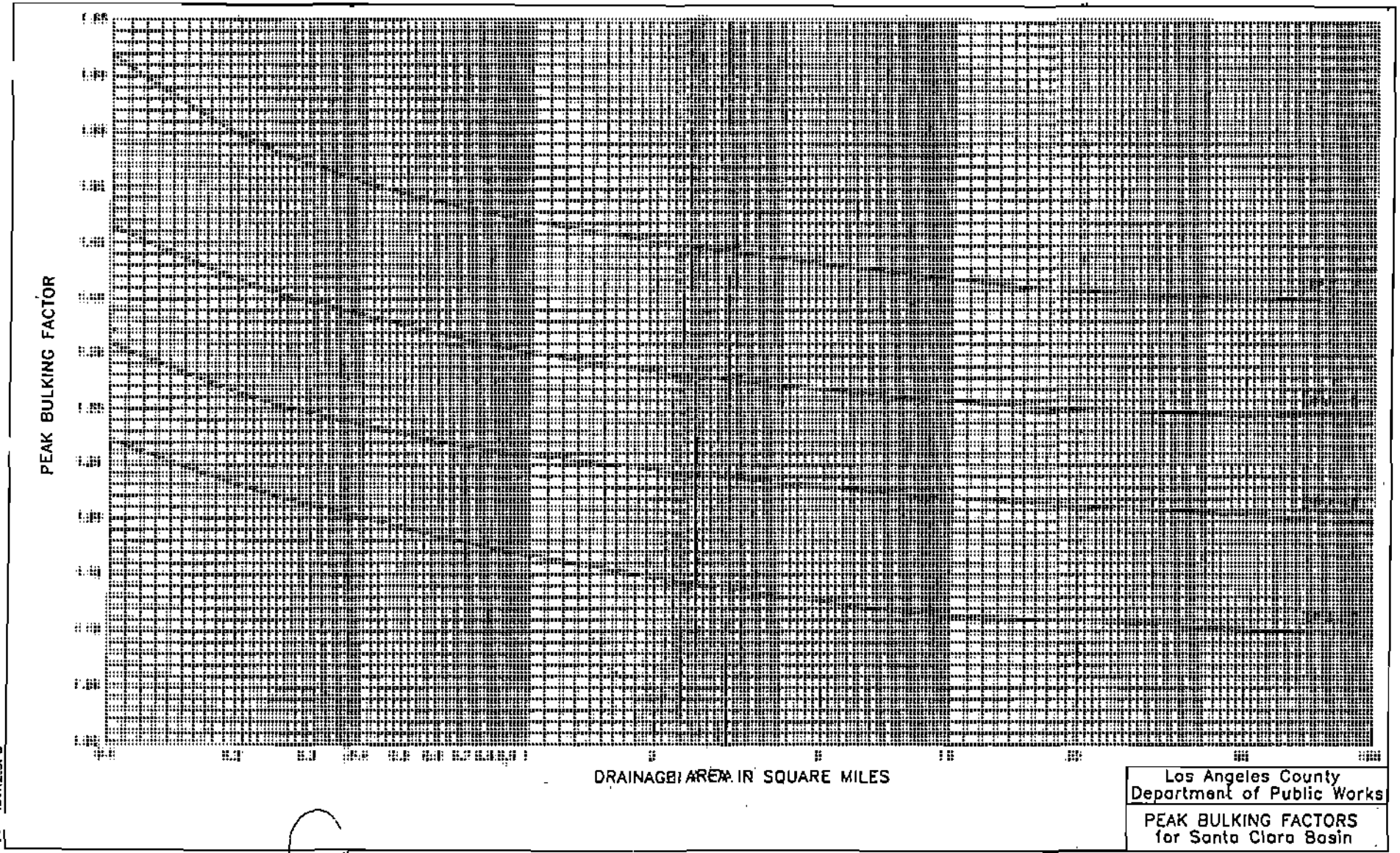
300 30A TA 1164 QA 455. QAC 470. QC 15. 300 30C TC 1161 QC 17. QCA 433. QA 416.
300 30AC TAC 1164 QAC 470. QA 455. QC 15.

LOCATION		SUBAREA AREA(Ac)	SUBAREA Q(CFS)	TOTAL AREA(Ac)	TOTAL Q(CFS)	CONV TYPE	CONV LNGTH(Ft)	CONV SLOPE	CONV SIZE(Ft)	CONV Z	CONTROL Q(CFS)	SOIL NAME	TC	RAIN ZONE	PCT IMPV
300	30AC	10.3	17.	349.4	470.	0	0.	.00000	.00	.00	0.	220	0	A29	.00
300	31A	.0	0.	349.4	470.	0	0.	.00000	.00	.00	0.	220	99	A29	.00

APPENDIX D

HYDROLOGY/SEDIMENTATION
LA COUNTY MANUAL P-2, P-5



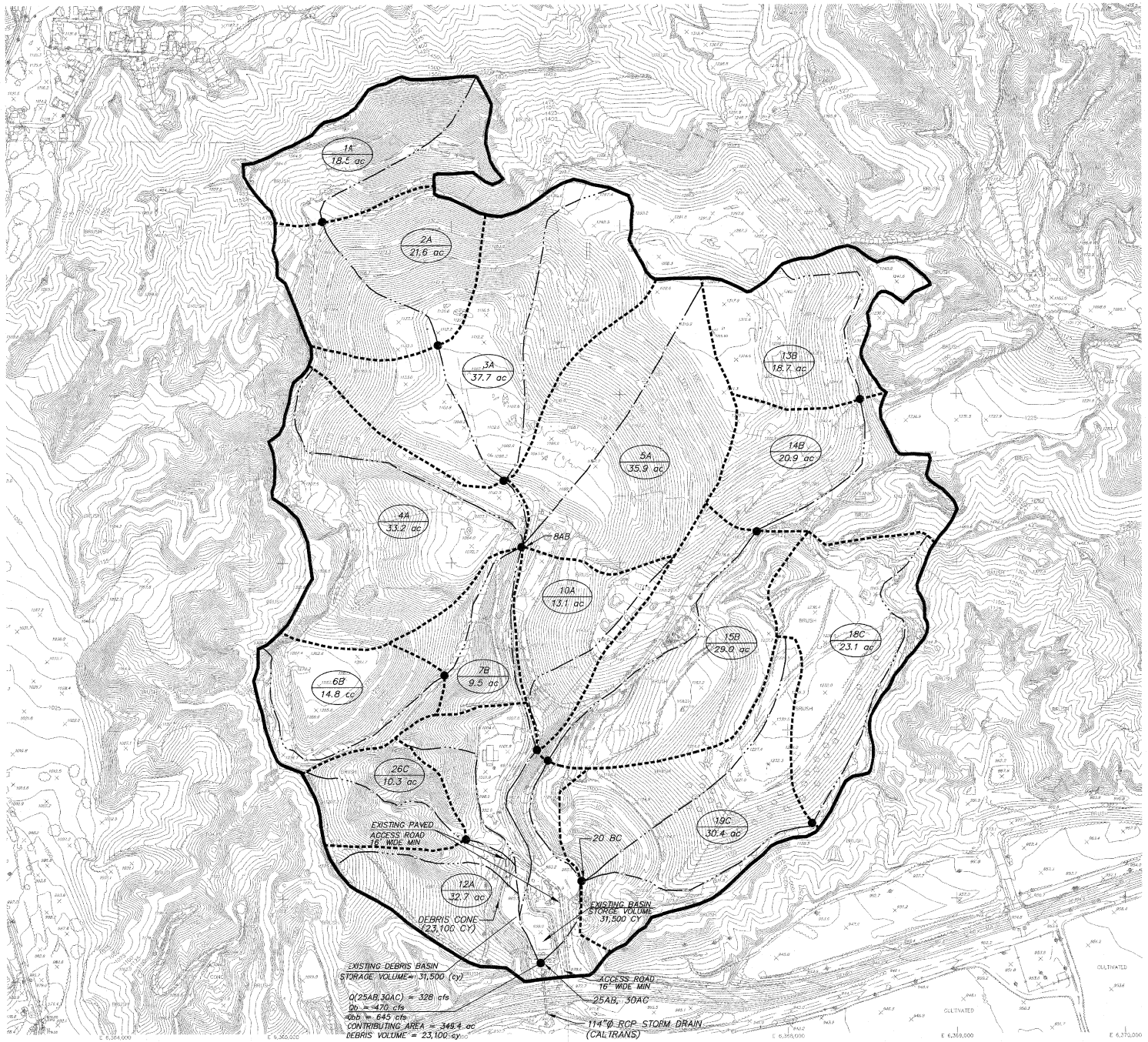


Los Angeles County
Department of Public Works

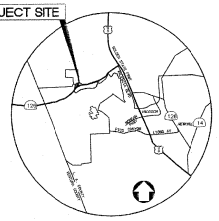
PEAK BULKING FACTORS
for Santa Clara Basin

PL :CVH2.SPG

APPENDIX E
HYDROLOGIC MAP



PROJECT SITE



VICINITY MAP

NOT TO SCALE

DEBRIS POTENTIAL AREA (DPA=5)

LEGEND:

4A / 4.4 ac — SUB-BASIN NUMBER / SUB-BASIN AREA (ac)

SUB-BASIN BOUNDARY

DRAINAGE AREA BOUNDARY

FLOWLINE/ROUTING REACH

CONCENTRATION POINT

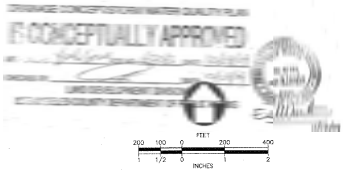
DRAINAGE CONCEPT/STORM WATER QUALITY PLAN
IS CONCEPTUALLY APPROVED
APPROVED BY: [signature] FOR NO. 67385 DATE: 11/29/06
CHECKED BY: _____ DATE: 11/29/06
LAND DEVELOPMENT DIVISION
LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

EXISTING CONDITION

SUB-BASIN NUMBER	INCREMENTAL SUB-BASINS: TIME OF CONCENTRATION (minutes)	SUB-BASIN AREA (acres)	Q50 UNBURNED (cfs)	Q50 BURNED (cfs)	Q50 BURNED + BULKED (cfs)	CUMULATIVE SUB-BASINS: A CUMULATIVE (cy)	DEBRIS PROD RATE (cy/sq mi)	DPV CUMULATIVE (cy)
1A	11	18.5	24	32	47	18.5	82,000	2,370
2A	9	21.6	32	42	61	40.1	82,000	5,138
3A	16	37.7	39	53	77	77.8	75,000	9,117
4A	16	33.2	35	46	68	111.0	54,000	11,100
5A	15	35.9	38	52	76	146.9	56,000	12,834
10A	13	13.1	15	20	30	160.0	54,000	13,500
12A	16	32.7	34	46	67	192.7	50,000	15,055
6B	10	14.8	16	21	31	207.5	49,000	15,887
7B	10	9.5	13	17	25	217.0	48,000	16,275
13B	12	18.7	23	31	45	235.7	47,000	17,309
14B	14	20.9	24	31	46	256.6	46,000	18,443
15B	17	29.0	29	39	57	285.6	45,000	20,081
18C	14	23.1	26	35	51	308.7	44,000	21,223
19C	18	30.4	32	42	62	339.1	43,000	22,783
26C	9	10.3	15	20	29	349.4	42,300	23,093
TOTAL:	---	349.4	328	470	645*	349.4	---	23,100**

* BULKING FACTOR IS TAKEN FROM HYDROLOGY/SEDIMENTATION APPENDIX, P-5

** DEBRIS PRODUCTION RATE IS TAKEN FROM P-2, DPA-5



PROJECT AREA

ROSS W. BARKER No. C-32799 REN. 06/30/08 CIVIL STATE OF CALIFORNIA	**PSOMAS** 28470 Avenue Stanford, Suite 300 Santa Clarita, CA 91355 (661) 259-6040 (661) 775-2778 (FAX) 11-17-2006 ROSS W. BARKER R.C.E. NO. C32799 DATE	DESIGNED : AC/EW DRAFTED : AC/ML CHECKED : SW/MG/RB	OWNER/DEVELOPER : NEWHALL RANCH 23823 WEST VALENCIA BOULEVARD VALENCIA, CALIFORNIA 91355 TELEPHONE: (661) 255-4000 REPRESENTATIVE : MR. FRED MACMURDO	**EXISTING DRAINAGE PLAN FOR CHIQUITO LANDFILL** IN THE UNINCORPORATED AREA OF THE COUNTY OF LOS ANGELES STATE OF CALIFORNIA	PROJECT NO. 1NRC01070B SHEET 1 OF 1

RVO90-01.dwg

The Complete
Landmark Village
Tentative Tract Map 53108
Drainage Concept Report
including appendices
is available on the attached CD.

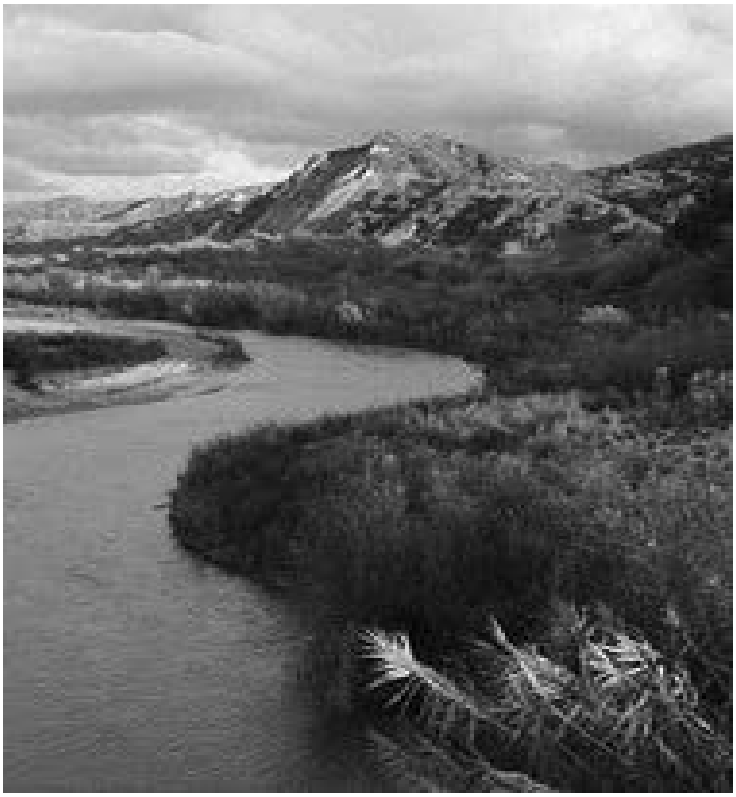
APPENDIX F4.3

Water Quality

UPPER SANTA CLARA RIVER

CHLORIDE TMDL RECONSIDERATION, ~~AND~~ CONDITIONAL SITE SPECIFIC OBJECTIVES FOR CHLORIDE, AND INTERIM WASTELOAD ALLOCATIONS FOR SULFATE AND TOTAL DISSOLVED SOLIDS

STAFF REPORT



CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD - LOS ANGELES REGION

~~September, 30~~November 24 2008

Executive Summary

Chloride levels in the upper Santa Clara River (USCR) and in nearby groundwater basins have increased over the past three decades due to increased salt loadings from water imported into the Santa Clarita Valley and the increased number of self regenerating water softeners in the Santa Clarita Valley. Since the 1970s, growth in the Santa Clarita Valley has lead to chloride levels that exceed the water quality objective and impair beneficial uses for agricultural supply. Agriculture is the largest industry in the Santa Clara River Valley and the Regional Board has adopted a TMDL to restore the Santa Clara River to attain its beneficial uses.

This Staff Report discusses efforts under the Upper Santa Clara River Chloride TMDL to address these impairments with particular emphasis on the recent studies which have lead to a stakeholder developed plan for complying with the TMDL. The stakeholder plan, termed "Alternative Water Resources Management Plan" (AWRM) considers the results of key TMDL studies on the chloride sensitivity of crops and aquatic life and the interaction of groundwater and surface water in the USCR to fashion a plan that provides reduction of chloride loads from current levels, enhancement of water supplies for recycling and downstream uses, restoration of groundwater basins underlying the Upper Santa Clara River, and consideration of critical conditions such as a sustained drought. The AWRM requires a revision to existing water quality objectives for chloride, but it provides a significant reduction in chloride loading from current levels such that the most stringent beneficial uses are attained. During the critical condition of sustained drought, growers are provided alternative water to meet requirements and the chloride exported from the watershed still exceeds chloride into the watershed so that groundwater conditions will continue to improve.

The Regional Board first adopted a Total Maximum Daily Load (TMDL) for chloride in the USCR in 2000. The TMDL showed that chloride is loaded primarily into the Santa Clara River from Water Reclamation Plants serving residential, commercial and industrial users in the Santa Clarita Valley. The sources of the chloride which are loaded into the SCR are primarily chloride contained in the imported source water and chloride added by domestic uses, including self regenerating water softeners. As the Santa Clarita Valley has grown over the past decades, these TMDL source analyses also showed that the water quality objectives could not be met with source control alone, and that some type of advanced treatment would be necessary.

The identification of remedies for chloride impairments is challenging due to stakeholders with widely different interests in Los Angeles and Ventura Counties and potentially costly implementation measures. These factors lead to a remand of the TMDL from State Water Resources Control Board and after reconsideration by the Regional Board, the TMDL became effective on May 5, 2005. Key provisions of this TMDL include special studies to address scientific uncertainties and a consideration of site specific objectives by the Regional Board. This Staff Report summarizes the results of the special studies and discussions with stakeholders, which lead to an AWRM program to comply with the TMDL. This report considers the antidegradation and Water

Code Section 13241 requirements and recommends conditional site specific objectives to implement the AWRM.

Prior to completion of the special studies, the presumed implementation plan included two options: advanced treatment of effluent from the Saugus and Valencia water reclamation plants and disposal of brine in a new ocean outfall or disposal of effluent from the Saugus and Valencia water reclamation plants in a new ocean outfall. Both options entail construction of a pipeline from the Santa Clarita Valley WRPs and an ocean outfall. Concerns regarding the cost and feasibility of constructing this line lead caused controversy amongst stakeholders.

The TMDL Special Studies, all conducted in a facilitated stakeholder process in which stakeholders in scoping and reviewing the studies addressed three scientific uncertainties: 1) the levels of chloride required to support irrigation of salt sensitive crops; 2) the interaction of surface water and groundwater and the fate and transport of chloride in the USCR; 3) the effects of chloride on threatened and endangered fish in the USCR.

Regional Board staff finds that the work to date provides sufficient information on the chloride hazard threshold for salt-sensitive crops, the chloride threshold for endangered species, and the hydraulic and contaminant interactions between surface waters and groundwater basins in the USCR watershed to demonstrate that conditional site specific objectives can be combined with reverse osmosis technology to effectively reduce chloride loadings to the USCR and protect beneficial uses. Completion of the Literature Review and Evaluation (LRE) provided a scientifically defensible baseline to support a Water Quality Objective (WQO) of 117 milligrams per liter (mg/L) that is protective of agricultural supply beneficial use (AGR). The endangered species study shows that the chloride threshold for protection of salt sensitive agriculture is also protective of threatened and endangered species. The groundwater surface water interaction model shows that surface flows in the river recharge the Piru Basin with attendant chloride accumulation in that groundwater Basin. The AWRM consists of chloride source reduction actions and chloride load reduction through advanced treatment of the Valencia WRP effluent in conjunction with conditional site specific objectives. These source and load reductions mitigate the effect of any chloride accumulation in the groundwater basin.

The TMDL provides a ten-year schedule to attain compliance with the conditional SSOs. Key uncertainties at this point relate to identification of the optimum method for brine disposal. Several options, including deep-well injection in the vicinity of old oil fields in the Santa Clarita Valley, and drying and landfill disposal will be considered by the Santa Clarita Sanitation District of Los Angeles County in the first two years of the TMDL Implementation Plan. The recommended water quality objective changes before the Board are conditioned on implementation of the AWRM program; if the AWRM system is not built, the water quality objectives revert back to the current levels in the Basin Plan.

Staff's recommendation is to adopt the conditional site specific objectives for chloride. Staff finds that the costs of implementing the AWRM program will not increase monthly sewage rates substantially above the state average and median rates. Staff notes that the existing TMDL schedule can be accelerated by one year from 11 years to 10 years.

List of Acronyms

AGR – Agricultural Supply Beneficial Use
AWRM – Alternative Water Resources Management
BPA – Basin Plan Amendment
cfs – cubic feet per second
CLWA – Castaic Lake Water Agency
EIR – Environmental Impact Report
ESA – Extended Study Alternatives
ESP – Endangered Species Protection
GWR – Groundwater Recharge Beneficial Use
GSWI – Groundwater and Surface Water Interaction Model
LA – load allocation
LRE – Literature Review and Evaluation
LWA – Larry Walker Associates
MCL – Maximum Contaminant Level
MF/RO – microfiltration-reverse osmosis
MGD – million gallons per day
mg/L – milligrams per liter
NPDES – National Pollutant Discharge Elimination System
O&M – operation and maintenance
ppd – pounds per day
RARE – Rare and Endangered Species Habitat Beneficial Use
RO – Reverse Osmosis
SARI – Santa Ana Regional Interceptor
SB475 – Senate Bill 475
SCV – Santa Clarita Valley
SCVJSS – Santa Clarita Valley Joint Sewerage System
SCVSD – Santa Clarita Valley Sanitation District of Los Angeles County
SRWS – Self-Regenerating Water Softener
SSO- Site Specific Objective
SWP – State Water Project
SWRCB – State Water Resources Control Board
TAP – Technical Advisory Panel
TDS – Total Dissolved Solids
TMDL – Total Maximum Daily Load
USBR – United States Bureau of Reclamation
USCR – Upper Santa Clara River
USEPA – United States Environmental Protection Agency
USGS – United States Geological Survey
UWCD – United Water Conservation District
WLA – Wasteload Allocation
WQO – Water Quality Objective
WRP – Water Reclamation Plant

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1. Introduction

This staff report discusses the scientific and regulatory basis for proposed Basin Plan amendments to revise the Upper Santa Clara River Chloride Total Maximum Daily Load (TMDL) and establish conditional site-specific water quality objectives (SSOs) for chloride in reaches and groundwater basins in the Upper Santa Clara River watershed.

The Los Angeles Regional Water Quality Control Board (Regional Board) adopted a TMDL to address chloride impairments of the USCR on July 10, 2003 (Resolution 03-008). On May 6, 2004, the Regional Board amended the USCR chloride TMDL to revise the interim wasteload allocations (WLAs) and implementation schedule (Resolution 04-004). The amended TMDL was approved by the State Water Resources Control Board (State Board), Office of Administrative Law and United States Environmental Protection Agency (USEPA), and became effective on May 4, 2005.

At the time the TMDL was adopted and approved, there were key scientific uncertainties regarding the sensitivity of crops to chloride and the complex interactions between surface water and groundwater in the Upper Santa Clara River watershed. However, the TMDL found that the chloride sources are primarily imported source water from the State Water Project and chloride added by domestic uses, including self regenerating water softeners. These chloride sources are loaded into the USCR in effluent from the Saugus and Valencia Water Reclamation Plants (WRPs) that serve residents and industries in the Santa Clarita Valley. The TMDL recognized the possibility of revised chloride water quality objectives (WQOs) and included mandatory reconsiderations by the Regional Board to consider SSOs. The TMDL required the Santa Clarita Valley Sanitation District of Los Angeles County (SCVSD[1]) to implement special studies and actions to reduce chloride loadings from the Saugus and Valencia WRPs. The TMDL included the following special studies to be considered by the Regional Board:

- Literature Review and Evaluation (LRE) – review agronomic literature to determine a chloride threshold for salt sensitive crops.
- Extended Study Alternatives (ESA) – identify agricultural studies, including schedules and costs, to refine the chloride threshold.
- Endangered Species Protection (ESP) – review available literature to determine chloride sensitivities of endangered species in the USCR.
- Groundwater and Surface Water Interaction Study (GSWI) – determine chloride transport and fate from surface waters to groundwater basins underlying the USCR.

[1]Prior to 2005, the Santa Clarita Valley was historically served by the County Sanitation District Number 26 of Los Angeles County (Saugus WRP) and County Sanitation District Number 32 of Los Angeles County (Valencia WRP). Both of these Districts were collectively referred to as the County Sanitation Districts of Los Angeles County or CSDLAC in previous documents related to the Upper Santa Clara River Chloride TMDL. These two districts were merged into a single district, the Santa Clarita Valley Sanitation District of Los Angeles County or SCVSD as of July 1, 2005.

- Conceptual Compliance Measures – identify potential chloride control measures and costs based on different hypothetical WQO and final WLA scenarios.
- Site Specific Objectives and Antidegradation Analysis - consider a site-specific objective for chloride based on the results of the agricultural chloride threshold study and the GSWI.

The TMDL special studies were conducted in a facilitated stakeholder process in which stakeholders participated in scoping and reviewing the studies. This process has lead stakeholders to develop an alternative TMDL implementation plan that addresses chloride impairment of surface waters and degradation of groundwater. The alternative, termed Alternative Water Resources Management (AWRM) was first set forth by Upper Basin water purveyors and United Water Conservation District (UWCD), the management agency for groundwater resources in the Ventura County portions of Upper Santa Clara River watershed.

This Staff Report first presents a background on the TMDL, including regulatory history, the stakeholder collaborative process, a description of the watershed and the sources of chloride, and other salinity management programs in the state. The report then discusses the results and conclusions of the special studies which led to the development of the AWRM Program and proposed conditional SSOs. The AWRM Program and the proposed conditional SSOs needed to support the AWRM are then discussed. The report then discusses one of the special studies in detail, the Site Specific Objectives/ Antidegradation Analysis, which provides the regulatory basis for the conditional SSOs. Finally, the staff report reviews the alternatives for TMDL implementation based on the results of the special studies, provides staff's recommendation for conditional SSOs and TMDL revisions, and discusses how the recommended conditional SSOs and TMDL revisions would be implemented.

2. Background

This section provides background information on chloride issues in the USCR watershed.

2.1. Regulatory History

The Regional Board has adopted several resolutions that regulated chloride in the USCR, starting with Resolution 75-21 in 1975, which established WQOs throughout the region.

In 1990, the Regional Board adopted the Drought Policy, Resolution 90-04. This resolution was intended to provide short-term and temporary relief to dischargers who were unable to comply with limits for chloride due to the effects of drought on chloride levels in supply waters imported to the Region. The Regional Board temporarily reset limits on concentration of chloride at the lesser of: (i) 250 mg/L, or (ii) the chloride concentration of supply water plus 85 mg/L. The Regional Board renewed the Drought Policy in 1993 and again in 1995 because the chloride levels in supply waters remained higher than the chloride levels before the onset of the drought. The Regional Board did not revise the chloride WQOs in the Santa Clara River and Calleguas Creek because of the potential to affect present and anticipated agricultural beneficial uses.

In 1997, the Regional Board adopted the Chloride Policy, Resolution No. 97-02. The Chloride Policy revised the chloride objective for the Los Angeles River, Rio Hondo, and San Gabriel River. Due to concerns expressed about the potential for future adverse impacts to agricultural resources in Ventura County, WQOs for chloride in the Santa Clara River and Calleguas Creek were not revised. Rather, the chloride policy provided surface water interim limits of 190 mg/L in the Santa Clara River that extended for three years following approval of the amendment. The Regional Board did not revise the chloride WQOs in the Santa Clara River and Calleguas Creek because of the potential to affect existing and anticipated AGR. Similarly, the Regional Board did not revise the groundwater objectives for chloride.

The Regional Board first adopted a TMDL for chloride in the USCR in October 2002 (Resolution No. 2002-018). The TMDL showed that the chloride sources are primarily chloride contained in the imported source water from the State Water Project and chloride added by domestic uses, including self regenerating water softeners. These chloride sources are loaded into the USCR in effluent from the Saugus and Valencia WRPs that serve residents and industries in the Santa Clarita Valley. The TMDL source analysis also showed that the water quality objectives could not be met with source control alone, and that some type of advanced treatment would be necessary. The TMDL contained an 8-1/2 year implementation plan to attain chloride WQOs.

Because of differing stakeholder interests and potentially costly implementation measures, the State Board remanded the Chloride TMDL (State Board Resolution No. 2003-0014) to the Regional Board in February 2003 due to concerns about the duration of the interim effluent limits and concerns that the original implementation plan could have required the SCVSD to embark on planning and construction of an advanced treatment even though such studies might have demonstrated a need that could have been proved unnecessary in the end. The remand resolution also directs the Regional Board to consider an integrated solution for all water quality pollutants in the SCR basin on the Clean Water Act 303(d) list. The Regional Board revised the TMDL Implementation Plan to extend the interim wasteload allocations and final compliance date to 13 years after the TMDL effective date. It also included two additional special studies and several mandatory reconsiderations of the TMDL by the Regional Board. The Regional Board adopted the revised TMDL in July 2003 (Resolution No. 2003-008).

The TMDL was amended in 2004 (Resolution No. 04-004) to conform the interim wasteload allocations for the Saugus and Valencia WRPs to the effluent limits in 1994 Time Schedule Orders associated with National Pollutant Discharge Elimination System (NPDES) permits. In May 2004, the Regional Board and SCVSD signed a Settlement Agreement and Stipulation Concerning Chlorides in the UCSR. The Regional Board and SCVSD agreed that, if or when new or revised NPDES permits are subsequently issued to the Saugus or Valencia treatment plants prior to the date that a revised WQO or final wasteload allocations take effect in accordance with the Chloride TMDL Amendments, interim chloride effluent limitations reflecting the interim wasteload allocations in the TMDL, including any revisions thereto, will be included in the revised permits.

In 2006, the Regional Board reconsidered the TMDL and amended the TMDL schedule. The Board considered the results of the special studies to date and found it appropriate to accelerate the study period of the Implementation Plan based on the Literature Review and Evaluation, which showed that the range of chloride values protective of AGR and GWR beneficial uses was significantly smaller than originally anticipated.

In 2007, the Regional Board amended the Basin Plan to divide Reach 4 into two separate reaches. This action was based on historical and current water quality, flow, and land use data showing significant water quality differences between the western and eastern portions of Reach 4. Staff found that Reach 4 of the SCR contains unique hydrogeologic conditions due to the significant alterations to land uses and waste discharges within the USCR watershed that supported the separation of the reach into two separate reaches, 4A and 4B, divided at the confluence of Piru Creek.

This proposed action represents the second Regional Board reconsideration of the TMDL, which is scheduled 3-years after the TMDL effective date. Specifically, Tasks 10.a and 10.d of the TMDL Implementation Schedule state, “Preparation and Consideration of a Basin Plan Amendment (BPA) to revise the chloride objective by the Regional Board” and “Reconsideration of and action taken on the Chloride TMDL and Final Wasteload Allocations for the Upper Santa Clara River by the Regional Board.”

2.2. Stakeholder Collaborative Process

Based on the Chloride Agreement and Stipulation discussed in Section 2.1, the Regional Board and the SCVSD entered into a collaborative process in June of 2004 to implement the TMDL special studies. The Regional Board and SCVSD have set up a facilitated process to allow for stakeholder input and review of the special studies as they are developed. The SCVSD, Regional Board, facilitators, consultants and stakeholders attended Technical Working Group meetings on a monthly basis in the Cities of Santa Clarita, Fillmore, and Santa Paula to discuss the TMDL special studies as well as other planning issues regarding chloride impairments within the Santa Clara River. About thirty people who represent a wide range of stakeholder interests, including Municipalities, County government, agricultural interests, water purveyors, and environmental interests, attend the meetings. There is a website, www.santaclarariver.org, which updates activities and progress on the USCR Chloride TMDL.

Additionally, an independent technical advisory panel (TAP) of recognized agricultural experts was engaged to review the results of the LRE. The TAP issued a separate report, which provides technical guidance on the use of the LRE for policy development. The TAP report largely confirmed the results of the LRE. Both the TAP Report and LRE are available to the public on the website listed above.

Finally, Regional Board staff has been meeting with SCVSD' staff and representatives of the Upper Basin Water Purveyers, UWCD, and Ventura County Agricultural Water Quality Coalition, to explore the potential implementation actions and site specific objectives for the TMDL. This process has lead to development of the AWRM and the development of proposed conditional SSOs to support the AWRM and protect beneficial uses.

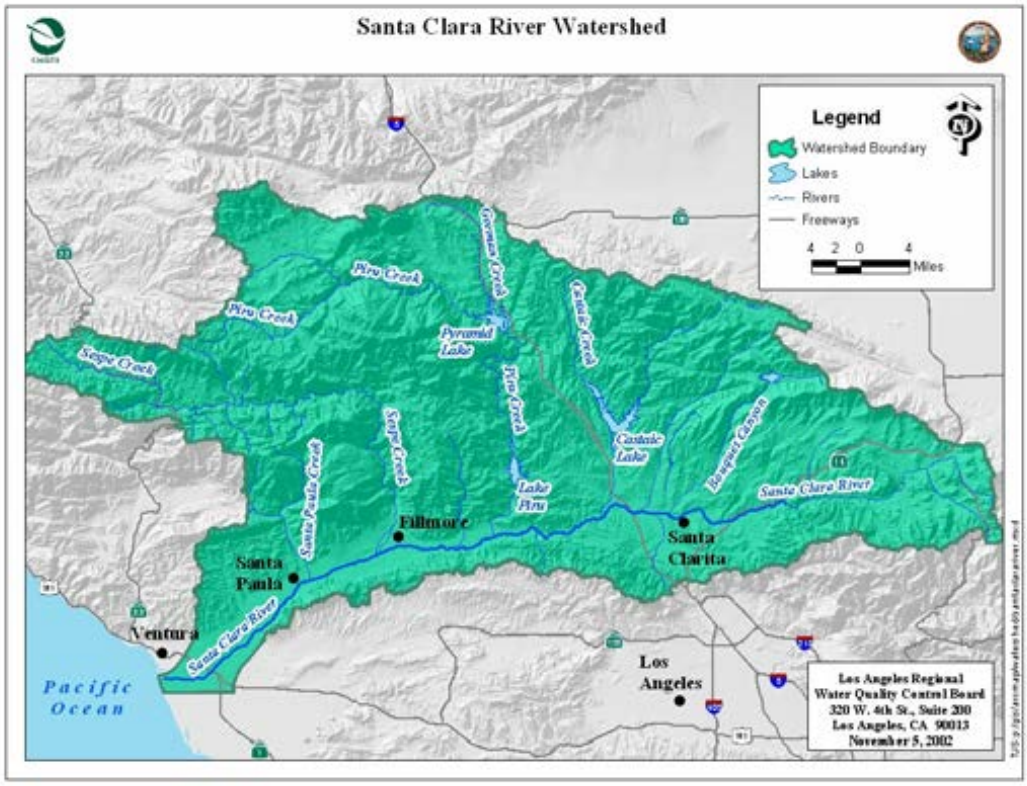
2.3. Environmental Setting

The Santa Clara River is the largest river system in Southern California that remains in a relatively natural state. The river originates on the northern slope of the San Gabriel Mountains in Los Angeles County, traverses Ventura County, and flows into the Pacific Ocean between the cities of San Buenaventura (Ventura) and Oxnard. Municipalities within the watershed include Santa Clarita, Newhall, Fillmore, Santa Paula, and Ventura (Figure 1).

Extensive patches of high quality riparian habitat exist along the length of the river and its tributaries. Two endangered fish, the unarmored stickleback and the steelhead trout, are resident in the river. One of the Santa Clara River's largest tributaries, Sespe Creek, is designated a wild trout stream by the state of California and a wild and scenic river by the United States Forest Service. Piru and Santa Paula Creeks, tributaries to the Santa Clara River, also support steelhead habitat. In addition, the river serves as an important wildlife corridor. The Santa Clara River drains to the Pacific Ocean through a lagoon that supports a large variety of wildlife.

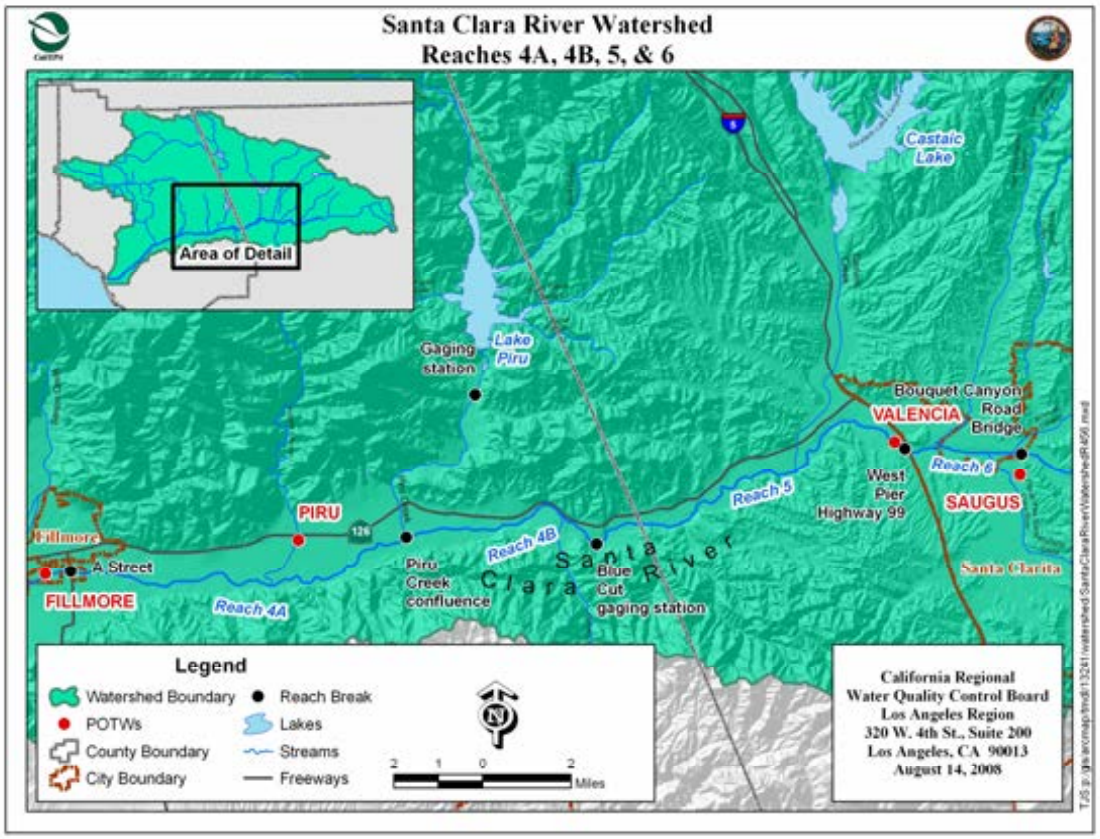
The predominant land uses in the Santa Clara River watershed include agriculture, open space, and residential uses. Revenue from the agricultural industry within the Santa Clara River watershed is estimated at over $700 million annually. Residential use is increasing rapidly both in the upper and lower watershed. The number of housing units in the watershed is estimated to increase by 187 percent from 1997 to 2025.

Figure 1. Santa Clara River Watershed



The upper reaches of the Santa Clara River include Reaches 5 and 6, which are located upstream of the Blue Cut gauging station, west of the Los Angeles - Ventura County line between the Cities of Fillmore and Santa Clarita. The upper boundary extends to Bouquet Canyon, upstream of the City of Santa Clarita. The portion of the river within Los Angeles County is generally described as the Upper Santa Clara River, and the portion within Ventura County is generally referred to as the Lower Santa Clara River. Two major point sources, the Saugus and Valencia WRPs, discharge to the USCR. Below Reach 5 are reaches 4A and 4B, divided at the confluence of Piru Creek (Figure 2).

Figure 2. Santa Clara River Watershed Reaches 4A, 4B, 5, and 6



2.4. Beneficial Uses and WQOs

Key beneficial uses and WQOs for the USCR are described in the Basin Plan and include agricultural supply (AGR), groundwater recharge (GWR) and rare and endangered species habitat (RARE). A full description of each of these beneficial uses is included in the Basin Plan. AGR is designated as existing or potential for all reaches of the Santa Clara River, including the USCR, except the headwaters. GWR is designated as an existing or potential beneficial use for the USCR. RARE is an existing and potential designated beneficial use for the upper reaches included in this TMDL. Two types of endangered and rare aquatic species are known to reside in the watershed: steelhead trout and unarmored three-spine stickleback.

The current WQO for chloride in Reaches 4A, 4B, 5 and 6 of the Santa Clara River is 100 milligrams per liter (mg/L). The groundwater quality objectives for the Santa Clara – Piru Creek area are: 200 mg/L chloride in the Upper area (above Lake Piru), 200 mg/L in the Lower area east of Piru Creek, and 100 mg/L west of Piru Creek.

2.5. Chloride Sources and Water Quality

This section summarizes chloride sources in the USCR watershed and projections of the effects of future growth and chloride reduction measures on the final WRPs effluent quality. Regional Board and SCVSD staff analyzed chloride sources in the USCR watershed in the 2002 Regional Board TMDL Staff Report and in the SCVSD's 2002, 2005, 2006 and 2007 chloride reports. These analyses utilized mass balance techniques to identify and quantify chloride loads from imported water and residential, commercial, and industrial sources.

The key findings from these reports include:

- The average chloride concentration in the USCR, as measured at the Blue Cut gauging station and at the Ventura/Los Angeles county line, was 131 mg/L in 2002 and 126 mg/L in 2003. The average chloride concentration at the Blue Cut gauging station frequently exceeds the WQO of 100 mg/L.

- The total chloride load from the Saugus and Valencia WRPs ranged from 23,500 pounds per day (ppd) to 28,500 ppd in 2001 through 2007.

- The WRP effluent chloride load is comprised of two main sources: chloride present in the imported water supply and chloride added by residents, businesses, and institutions in the Saugus and Valencia WRP service area. The chloride load added by users can be further divided into two parts: brine discharge from self-regenerating water softeners (SRWSs) and all other loads added by users. Excluding the imported chloride load that exists in the water supply, non-SRWS sources of chloride include: residential, commercial, industrial, infiltration, and wastewater disinfection. The two largest sources of chloride in the WRP effluent are the imported water supply and SRWSs, which have historically comprised from 37% to 45% and from 26% to 33% of the chloride in the WRP effluent, respectively.

- Municipal supply in Santa Clarita Valley (SCV) water supply is a blend of State Water Project (SWP) water and local groundwater. Over the past 30 years, chloride concentrations in water from the SWP ranged from 28 mg/L to 128 mg/L. The quantity of SWP water served by SCV water purveyors has increased from 41,768 acre-feet in 2002 to 47,205 acre-feet in 2004. The use of imported water has grown steadily. As reported by the Castaic Lake Water Agency (CLWA), the use of SWP water by SCV water purveyors is projected to grow to 69,500 acre-feet by 2015.

- The chloride loads from SRWSs increased markedly from 1997 to 2003, when a ban on residential SRWSs was struck down by legislative action in 1997. A prospective ban on installation of new SRWSs was reinstated in 2003. The SCVSD reported a sharp decline in residential SRWS chloride contribution from 66 mg/L in 2004 to 35 mg/L during the first half of 2007. This large change in chloride loading represents the removal or inactivation of roughly 2,200 SRWSs, from a high in 2004 of 6,800 to 4,600 by July of 2007.

- In 2006, The SCVSD and the City of Santa Clarita co-sponsored Senate Bill 475 (SB475), which is authored by Senator George Runner of the 17th Senate District. SB 475 provides the SCVSD with the authority to require removal all SRWS remaining in the Santa Clara Valley that were installed prior to SCVSD's 2003 ordinance. SB 475 also includes establishments of a phased voluntary and mandatory program to compensate residents for the reasonable value and cost of removal and disposal of SRWS. SB 475 was passed by the Legislature on August 31, 2006, and signed into law on September 22, 2006. The SCVSD has enacted a new ordinance on June 11, 2008 banning the use of existing SRWS, which will become effective on January 1, 2009, contingent upon voter approval by the qualified voters in the SCVSD's service area. This ordinance will be considered for voter approval by qualified voters in the district's service area in the November 2008 general election.

The relative magnitude of chloride loads from different sources is summarized below:

Table 1. Relative Chloride Loadings to Saugus and Valencia WRPs Effluent by Source

Year	Water Supply	Ind.	Com.	Residential Non-SRWS	Residential SRWS	Inf.	Disinf.	Total Load
2001	42%	3%	4%	14%	33%	0%	4%	100%
2002	45%	2%	3%	13%	29%	0%	8%	100%
2003	45%	1%	3%	13%	31%	0%	7%	100%
2004	41%	1%	3%	14%	33%	0%	8%	100%
2005	37%	2%	3%	16%	30%	3%	9%	100%
2006	42%	2%	3%	18%	26%	0%	9%	100%
2007 (through June)	43%	2%	4%	17%	26%	0%	8%	100%

Note: Ind. indicates Industrial, Com. indicates Commercial, Inf. indicates Infiltration, Disinf. indicates Disinfection

2.6. Future Growth

Presently, there is extensive residential growth planned for the USCR watershed over the next several decades. The population of the SCV is growing very rapidly. The City of Santa Clarita is projected to grow from 151,800 residents in 2000 to 243,104 residents in 2010. The SCVSD estimates effluent flow from wastewater treatment plants will grow from approximately 20 million gallons per day (MGD) presently to about ~~32~~ 34 MGD ~~in~~ by 2027~~30~~. The effects of this growth on the chloride levels in the Santa Clara River and underlying aquifers were investigated through GSWI Study (see Section 3.4).

The Landmark Village project site is located in unincorporated Los Angeles County, within the SCV. The project site is located along the SCR, immediately west of the confluence of Castaic Creek and the SCR. The county line forms the western

boundary. The SCR forms the southern boundary of the project site, while the northern project boundary is defined by State Route 126. The project applicant proposes to develop the 292.6-acre Landmark Village tract map site, located in the first phase of the Riverwood Village within the boundary of the approved Newhall Ranch Specific Plan. The Landmark Village tract map site proposes construction of 1,444 residential dwelling units, 1,033,000 square feet of mixed-use/commercial uses, a 9-acre elementary school, a 16-acre community park, public and private recreational facilities, trails, and road improvements. Several off-site project-related components would also be developed on an additional 679.2 acres of land. The project also includes a 6.8 MGD WRP (Newhall Ranch WRP) as associated facility (Impact Sciences, Inc., 2006).

Projections of future chloride loading to the USCR are dependent on several factors. Most importantly, the chloride contribution from the blended water supply varies greatly according to hydrologic conditions in Northern California because the salinity of SWP is dependent on the mix of fresh and brackish water in the San Francisco Bay – Delta which is the source of the water imported into the SCV. The timing and duration of future droughts are uncertain but based on review of more than thirty years of water quality data it is not unreasonable to conclude that California will experience several droughts within the next few decades.

Staff notes that growth within the SCV is accompanied by increasing demand for imported water and increasing chloride loads. In 1980, imported SWP comprised 1,125 acre-feet, approximately 5% of the total water supply to the SCV. By 1998, imported SWP comprised approximately 20,000 acre-feet, approximately 50% of the total water supply to the SCV.

Additionally, staff notes that the SCVSD's chloride report indicates that that chloride loading from non-SRWS residential sources in terms of ppd has been increasing. This increase is likely correlated with residential growth and increased residential wastewater flow and increased demand on water resources. The chloride load from non-SRWS residential sources increased from 3,562 ppd in 2002 to 4,272 ppd in 2006.

2.7. Salinity Management – Recent State and Regional Boards Actions

Water quality impairments by salts and chloride are a statewide issue. This section provides a brief overview of several current issues addressed by the State Board and the Central Valley, Santa Ana, and Los Angeles Regional Boards. It also reviews the status of salinity implementation activities in Northern California.

In the Central Valley region, salts in surface and ground water are largely derived from supply water from the SWP and the Delta Mendota Canal and from surface soil. Salinity impairments are exacerbated locally by other sources, such as discharges to land associated with municipal wastewater disposal. The Central Valley Regional Board has adopted several approaches for basin management within their jurisdiction. The Central Valley Regional Board established a policy to control groundwater degradation for the Tulare Basin, a policy to promote the maximum export of salt from the San Joaquin River

Basin, and a policy to control point source discharges to the Sacramento River Basin. At this time, salinity TMDL for the San Joaquin River has been developed to meet the objectives at Vernalis and a second phase of this TMDL is being developed for upstream stretches of the river. Further, the State Board may consider whether to adopt Cease and Desist Orders against the United States Bureau of Reclamation (USBR) and the Department of Water Resources with regard to their potential violation of conditions in their water right permits that require the USBR and the California Department of Water Resources to meet salinity standards in the Southern Delta.

In southern California, the USBR led a comprehensive regional salinity management study in support of the Southern California Water Recycling Projects Initiative. The study was conducted by CH2M Hill and identified a range of projected brine discharge volumes for Southern California. Some of the factors influencing this projected range are the salinity of imported water, the stringency of wastewater effluent regulation, and the level of seawater desalting. The study predicted a regional brine discharge volume ranging from 43.7 MGD to 2,011 MGD. In addition to predicting future brine discharge volumes, the study identified the location of existing and potential future brine/concentrate management facilities in southern California. These facilities include 86 pipelines, 113 wastewater treatment plants, 32 groundwater desalters, 9 seawater desalination facilities, and 9 major groundwater basins (with 91 sub-basins).

An established Southern California salinity management facility is the Arlington Desalter Facility and the Santa Ana Regional Interceptor (SARI). The Desalter, using Reverse Osmosis (RO) technology, produces up to 6 MGD of blended desalinized water, with another estimated 1 MGD of concentrated brine generated by the plant discharged to the SARI line. The SARI line, a regional brine line, is designed to convey 30 MGD of non-reclaimable wastewater from the upper Santa Ana River basin to the ocean for disposal, after treatment. The non-reclaimable wastewater consists of Desalter concentrate and industrial wastewater. Domestic wastewater is also received on a temporary basis. To date over 73 miles of the SARI line have been completed. The most recent extension (23 miles in length), the Temescal Valley Regional Interceptor line was completed in 2002. The upstream extension was completed in 1995 to the City of San Bernardino Wastewater Treatment Plant. The SARI also serves the Chino Basin area and the City of Riverside.

Desalinization treatment facilities have been planed in several regions of the state. The Northern California Salinity Coalition is planning RO treatment facilities to draw and treat water with a high salinity concentration from shallow aquifers in order to reduce net salt loading in groundwater basins of the Bay Area. The USBR proposed using RO to treat reused drainage water from an agricultural subsurface drainage system in the San Luis and Northerly Area of the Central Valley. Drainage will be collected from the fields and sent to one of 16 reuse areas to irrigate salt tolerant crops. The drainage from the reuse areas will then be collected and sent to Point Estero for ocean disposal or to a treatment facility.

Staff also notes that within the Region, the City of Los Angeles has implemented a RO facility at the Terminal Island Treatment Plant in order to meet local water quality targets. The facility processes 4.5 MGD and produces potable water for injection to the seawater barrier in the Dominguez Gap. The reverse osmosis effluent meets standards established by the Department of Health Services and is suitable not only for injecting into groundwater basins but also as boiler feed water for local industries.

In 2006, the Los Angeles Regional Board adopted the Calleguas Creek Watershed Salts TMDL based on a salts balance for that watershed. The Regional Board found that the water quality impairments and groundwater degradation in the Calleguas Creek watershed are due to a greater mass of salts imported to the watershed than exported from the watershed. The TMDL requires salt export throughout the watershed to achieve a salt balance, reduce salt load to surface and groundwater, and achieve and maintain water quality objectives for salts in the watershed. The Calleguas Creek watershed TMDL Implementation Plan is based on construction of a regional brine line and ocean outfall through which brines from the advanced treatment of degraded groundwater in the Calleguas Creek watershed are discharged directly to the ocean in compliance with the state Ocean Plan. The TMDL implementation plan also includes increased use of POTW effluent and advanced treated (reverse osmosis) groundwater for recycled water use. This plan has collateral benefits of increasing local sources of water supply in the watershed.

3. Results of TMDL Special Studies

This section describes the results of TMDL Special Studies and other chloride management activities in the USCR watershed, which were considered by staff in proposing TMDL revisions and conditional SSOs for the USCR watershed.

3.1. Literature Review and Evaluation

The first TMDL special study, the LRE, was completed in September 2005 and presented to the Regional Board on November 3, 2005. The LRE reviewed approximately 200 technical articles on the chloride and salinity sensitivities of avocado, strawberry and nursery plants. The LRE found a guideline concentration range for chloride sensitivity for avocado of 100 –117 mg/L. There is not sufficient technical literature to determine a guideline range for strawberry and nursery crops. The LRE concluded that a conservative guideline concentration for chloride hazard is 100-117 mg/L. The LRE was reviewed by an independent TAP and the majority TAP opinion concurred with the 100 –117 mg/L guideline concentration range. One minority TAP opinion advocated a higher guideline concentration and another minority TAP opinion recommended a maximum guideline concentration of 100 mg/L. As a supplement to the LRE, a memorandum on averaging period analysis was prepared by Newfields Agricultural and Environmental Resources (Newfields), in consultation with the TAP co-chairs, to determine what the applicable compliance averaging periods are for the LRE guideline concentration. The memorandum found that the minimum time between the beginning of exposure to chloride stress and signs of visible leaf chloride injury is between 2 and 9 weeks when high chloride concentrations are applied (at least 170 mg/L), and usually between 4 and 8 weeks. Based on an analysis of the literature and the receiving water variability, a three-month averaging period was recommended. (Newfields, 2008)

3.2. Extended Study Alternatives

This task provided an overview of the types of agricultural studies that are available to further define an appropriate threshold for protection of AGR in the Santa Clara River Watershed. The ESA evaluated study options ranging from surveys to field experiments and estimated a period of 2 to 10 years to develop adequate local data to define a site-specific threshold different from the threshold determined by the LRE. The ESA also documented the complexities of determining the effects of chloride on crop productivity under field conditions. Staff finds that the duration of time and the treatments proposed by the ESA might not be sufficient to address all the factors that may affect the chloride threshold level, and, absent a lengthy TMDL schedule extension, might not provide conclusive data to meet the TMDL requirements.

3.3. Endangered Species Protection

This task provided a review of technical literature regarding the chloride sensitivity of several endangered aquatic and riparian species to better understand the potential exposure and tolerance of these species to chlorides in the USCR. Special attention was given to resident species including Unarmored Three-Spine Stickleback, Steelhead Trout, Arroyo Toad, Red-Legged Frog and Cottonwood tree. Evaluation of overall toxicity data indicates that chloride concentrations for acute and chronic toxicity would be fully protective of Threatened and Endangered species in the USCR. Thus, the existing US EPA chronic chloride criteria of 230 mg/L can be considered to be fully protective of local biota. These conclusions indicate that endangered species can tolerate higher levels of chloride than salt-sensitive agricultural crops. The study results were reviewed by an independent TAP with the TAP finding the report supports the conclusion that the existing US EPA criteria are protective of threatened and endangered species in the Santa Clara River.

3.4. Groundwater and Surface Water Interaction Model

The GSWI model study was developed to determine the linkage between surface water and groundwater quality with respect to chloride and total dissolved solids (TDS) in the USCR. The model simulated historical water levels, flows, and concentrations and movement of chloride in surface water and groundwater in the USCR watershed from 1975 through 2005. The calibrated model was reviewed and approved as an appropriate and adequate modeling tool by the stakeholders and an independent GSWI TAP. The model was then used to assess the assimilative capacity of the surface water in Reaches 4, 5 and 6 and the groundwater basins underlying those reaches. The model was also used to determine the gradient of chloride concentrations from the Saugus and Valencia WRP outfalls to downstream receiving water stations and to assess the impacts of WRP effluent on underlying groundwater in the USCR. The model was then used to simulate future potential chloride impacts from 2007 to 2030 based on various combinations of high, intermediate and low reuse of recycled water from the with various levels of advanced treatment or SRWS removal rates. The results of the initial GSWI study are presented in a report entitled “Task 2B-1 Numerical Model Development and Scenario Results” (CH2M Hill, 2008; Geomatrix, 2008a).

Based on the model, none of the alternatives were predicted to comply with the existing chloride WQO of 100 mg/L at all times and at all locations (Table 2).

Table 2. Attainment Frequencies of Compliance Options-Existing Water Quality Objective

Compliance Options	Surface Water at Blue Cut Reach 4B Surface Water WQO 100 mg/L	East Piru Basin Groundwater Reach 4B Surface Water WQO 100 mg/L	East Piru Basin Groundwater Reach 4B Ground-water WQO 200 mg/L	West Piru Basin Groundwater Reach 4A Surface Water WQO 100 mg/L	West Piru Basin Groundwater Reach 4A Ground-water WQO 100 mg/L
Advanced Treatment	66.8	55.0	100.0	100.0	100.0
Minimal Discharge	65.5	62.1	100.0	100.0	100.0
Zero Discharge	63.8	68.3	100.0	100.0	100.0
Alternate WRP Discharge Location	48.9	46.1	100.0	100.0	100.0
AWRM	43.5	56.3	100.0	100.0	100.0

Note: Values represents percentage of days during simulation period that chloride is predicted to be equal to or less than the WQO concentration

Only the advanced treatment scenarios would produce surface water chloride concentrations less than the upper bound of the LRE chloride threshold of 120 mg/L (Table 3).

Table 3. Attainment Frequencies of the Compliance Options-LRE Water Quality Objective

Compliance Options	Surface Water at Blue Cut Reach 4B Surface Water WQO 120 mg/L	East Piru Basin Groundwater Reach 4B Surface Water WQO 120 mg/L	East Piru Basin Groundwater Reach 4B Ground-water WQO 200 mg/L	West Piru Basin Groundwater Reach 4A Surface Water WQO 120 mg/L	West Piru Basin Groundwater Reach 4A Ground-water WQO 100 mg/L
Advanced Treatment	99.0	99.6	100.0	100.0	100.0
Minimal Discharge	87.8	98.8	100.0	100.0	100.0
Zero Discharge	80.7	97.5	100.0	100.0	100.0
Alternate WRP Discharge Location	76.0	80.5	100.0	100.0	100.0
AWRM	88.0	93.0	100.0	100.0	100.0

Note: Values represents percentage of days during simulation period that chloride is predicted to be equal to or less than the WQO concentration

As a result, stakeholders in the USCR developed the AWRM Program, which increases chloride WQOs in certain groundwater basins and reaches of the USCR watershed, decreases the chloride objectives in the eastern Piru Basin, and results in an overall reduction in chloride loading as well as water supply benefits.

3.5. Conceptual Compliance Measures (AWRM)

The GSWI model was used to assess the ability of the AWRM to achieve compliance with proposed conditional SSOs under future water use scenarios within the USCR watershed. The model was based on design capacities at Valencia WRP and Saugus WRP of 27.6 MGD and 6.5 MGD, for a total system design capacity of 34.1 MGD by year 2027. The results of this effort are presented in a report entitled "Task 2B-2 Assessment of Alternatives for Compliance Options Using the Groundwater/Surface Water Interaction Model" (Geomatrix, 2008b). The model predicted that the AWRM could achieve proposed conditional SSOs for chloride under both drought and non-drought conditions (Table 4).

Table 4. Attainment Frequencies of the AWRM Compliance Option for Revised WQO

Compliance Options	Reach 4B (at Blue Cut) Surface Water WQO 117 mg/L	Surface Water WQO 130 mg/L	Ground-water WQO 150 mg/L	Reach 5 Surface Water WQO 150 mg/L	Ground-water WQO 150 mg/L	Reach 6 Surface Water WQO 150 mg/L	Ground-water WQO 150 mg/L
AWRM Alternative	99.9	99.2	100.0	98.3-99.7	100.0	98.6-99.7	100.0

Note: Values represents percentage of days during simulation period that chloride is predicted to be equal to or less than the WQO concentration

3.6. Site Specific Objectives and Antidegradation Analysis

The Site Specific Objectives and Antidegradation analysis has been completed and is included in a report entitled "Task 7 and 8 Report Site Specific Objective and Antidegradation Analysis" prepared by Larry Walker Associates (LWA). This report also presents the costs associated with the AWRM compliance alternatives identified in the GSWI reports. The report found that adoption of proposed conditional SSOs, when implemented with the AWRM Program, would be consistent with the state and federal antidegradation policies. The results of the SSO/Antidegradation analysis are discussed further in Sections 6 and 7.

4. Alternative Water Resources Management Program

The AWRM Program is a result of joint efforts of the Upper Basin Water Purveyors[2], Ventura County agricultural and water interests[3], and the SCVSD to find a regional watershed solution for compliance with the TMDL that benefits parties in both Los Angeles and Ventura Counties. The AWRM Program, which is described in detail in the GSWI Task 2B-2 Report (Geomatrix, 2008b), consists of advanced treatment for a portion of the recycled water from the SCVSD's Valencia WRP, constructing a well field in the eastern Piru basin to pump out higher chloride groundwater, discharging the blended pumped groundwater and advanced treated recycled water to Reach 4A at the western end of the Piru basin at a chloride concentration not to exceed 95 mg/L (Reach 4A WQO is 100 mg/L), and providing supplemental water and advanced treated recycled water to the river.

The objectives of the AWRM program are to lower chloride concentrations crossing the County Line, comply with conditional SSOs, protect agricultural water users in the eastern Piru basin, mitigate high-chloride groundwater in the eastern Piru basin, and maximize water resources in Ventura County. The key elements of the AWRM Program focus on reducing chloride in the water reclamation plant effluent through:

- SRWS removal
- Conversion of treated wastewater disinfection from chlorine injection to ultra-violet light disinfection
- Construction of 3 MGD microfiltration-reverse osmosis (MF/RO) facility at the Valencia WRP
- Brine disposal via deep well injection
- Groundwater extraction from the Piru Basin
- Discharges of blended MF/RO water and extracted groundwater in Reaches 4A and 4B

These facilities would typically be operated in two modes depending on the SCVSD's ability to comply with applicable water quality objectives, which is correlated to ~~the~~ chloride concentrations in the State Water Project (SWP) supply water (Figure 3~~2~~). During typical hydrologic cycles, when the supply water concentration is below 80 mg/l, the SCVSD WRPs would be able to comply with applicable water quality objectives a majority of the time without having to discharge ~~the~~ RO permeate produced at the Valencia WRP to the Santa Clara River. Under these conditions, the RO permeate

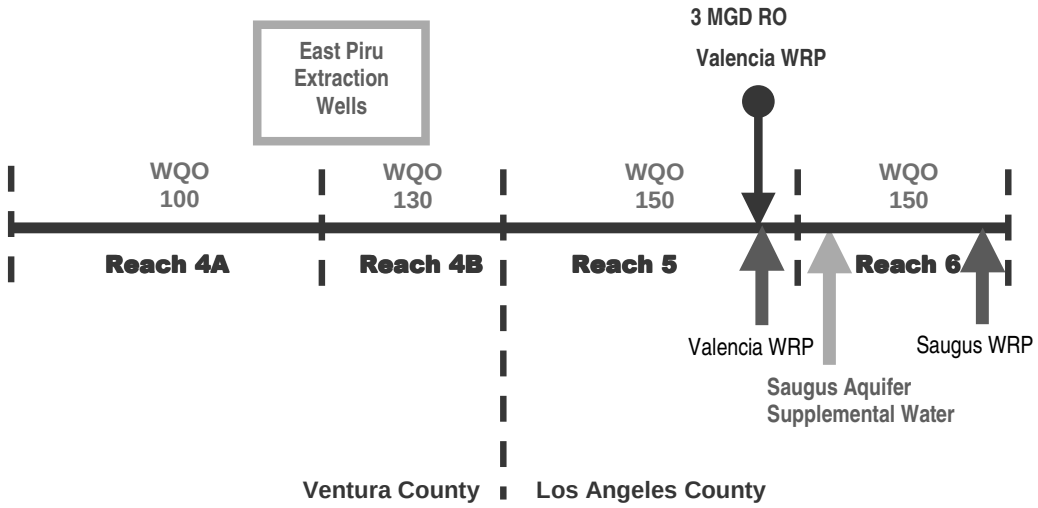
[2] The Upper Basin Water Purveyors are the Castaic Lake Water Agency, Valencia Water Company, Newhall County Water District, Los Angeles County Water Works District No. 36, and the Santa Clarita Water Division of the Castaic Lake Water Agency

[3] Represented by Ventura County Agricultural Water Quality Coalition (VCAWQC) and UWCD

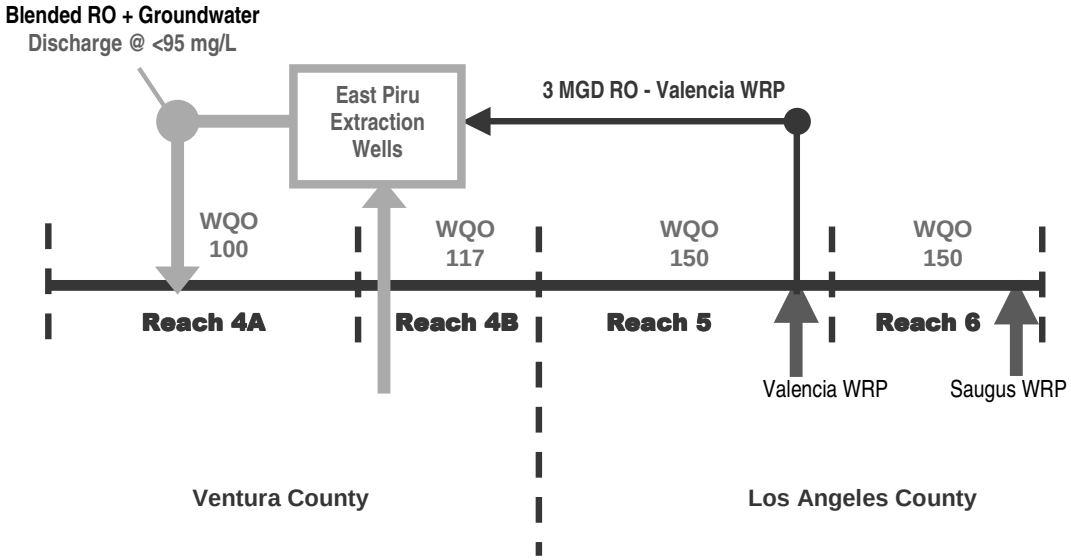
cwould be delivered to the extraction wells, blended with pumped groundwater, and discharged to Reach 4A for Ventura County water supply benefit. This option provides further water quality benefits for Ventura County because increased flows can mitigate sea water intrusion to the Oxnard Plain. During periods when the supply water concentration is above 80 mg/l, is typically when most, if not all of the RO permeate will be need to be discharged directly to 4B the Santa Clara River to comply with applicable water quality objectives. In addition some supplemental water would also be discharged as necessary to the Santa Clara River to reduce chloride concentrations in Reach 4B and comply with applicable water quality objectives.

Figure 32. Schematic of AWRM Facilities

Typical AWRM facility operation to comply with WQOs, when SWP > 80 mg/L



Typical AWRM facility operation to comply with WQOs, when SWP < 80 mg/L



Stakeholders have agreed upon the primary objectives for the uses of RO permeate from the MF/RO facility at the Valencia WRP. The primary objectives are prioritized as follows:

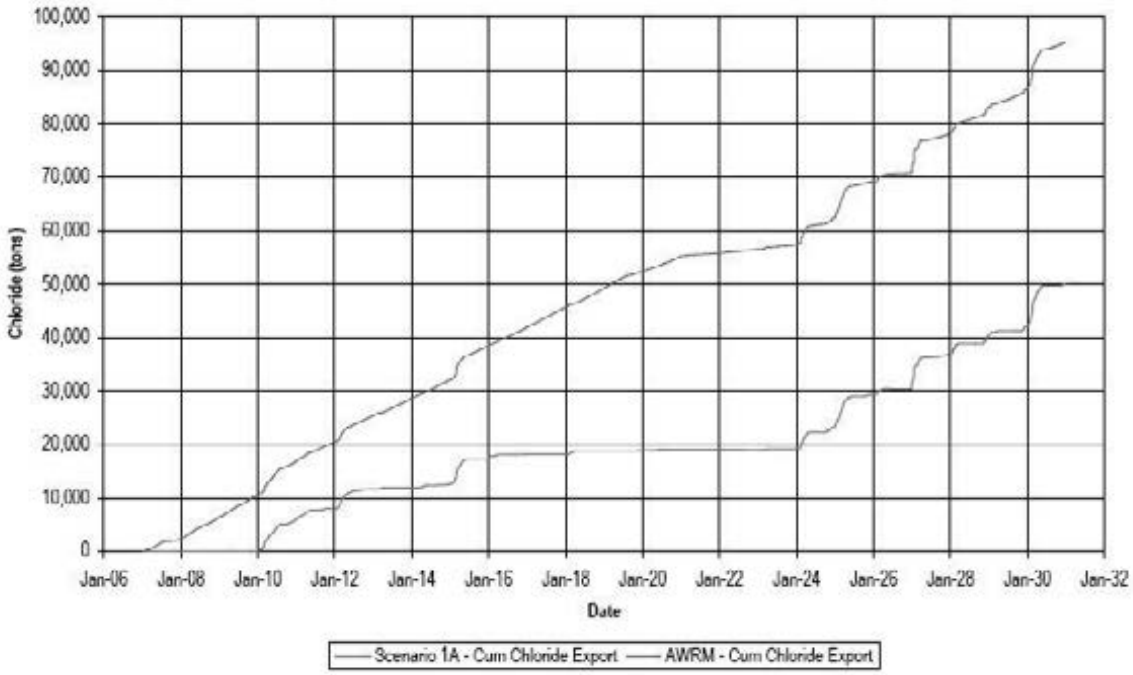
1) Compliance with conditional SSOs in the Santa Clara River at the County Line.
2) Provide alternative water supply to Camulos Ranch.
3) Achieve salt-balance in East Piru groundwater basin for past loading from surface water greater than 117 mg/L.
4) Achieve salt-balance in East Piru groundwater basin for any future loading from surface water greater than 117 mg/L.

The effects of the AWRM on surface water and groundwater have been evaluated using several tools. For Reaches 4B, 5, and 6 and the Piru basin, the primary tool was the GSWI model. Using the GSWI model, the AWRM has been shown to provide multiple water resource benefits, including:

- Increased flows in reaches 4A and downstream reaches of the USCR
- Improvement of groundwater quality in the Eastern Piru Basin
- Increased availability of irrigation and barrier water

The results of the GSWI model were used to calculate a mass balance to compare the predicted amount of salt exported under the AWRM compliance option with the predicted amount of salt exported under other compliance options to demonstrate the benefits to the East Piru Basin under the AWRM. Figure 4~~3~~ illustrates the cumulative salt export capabilities of the AWRM compliance option compared with the salt export capabilities of a maximum advanced treatment compliance option to meet the 100 mg/L chloride WQO (Scenario 1A).

Figure 4~~3~~. Cumulative Chloride Mass Export from East Piru Groundwater Basin: AWRM Option vs. Advanced Treatment Option (Scenario 1A)



Additionally, a study was prepared analyzing the effects of the AWRM Program in Ventura County (Bachman, 2008). The report found that the lowering of chloride concentrations in Reach 4B results in improved quality of water recharged to the East Piru Basin. Additionally, high chloride water that is pumped from the basin is recharged by lower chloride water during wet years. Using output from the GSWI model, UWCD's routing and percolation model was used to predict increased yield at the Freeman Diversion from implementation of the AWRM Program. The difference in yield at the Freeman Diversion between the Minimum Discharge option and the AWRM option is 11,500 AFY, which is approximately double the increased yield of 6,000 AFY when the permanent Freeman Diversion was constructed. This could result in a significant decrease in saline intrusion in the Oxnard Plain.

4.1. Conditional Site Specific Objectives to Support AWRM

The AWRM compliance option provides greater benefits than other potential scenarios and compliance options that have been identified. However, it will not result in compliance with the 100 mg/L water quality objectives at all times and in all locations for Reaches 4B, 5 and 6 of the USCR. Given the benefits of chloride reduction and protectiveness of the AWRM compliance option and in the context of achieving a salt balance for the watershed and protecting beneficial uses, staff proposes conditional SSOs that support the AWRM, while still being protective of beneficial uses (see Sections 5 and 6). Conditional SSOs for surface water and groundwater are presented in Tables 5 and 6. These conditional SSOs shall apply and supersede the existing regional water quality objectives of 100 mg/L only when chloride load reductions and/or chloride export

projects are in operation by the SCVSD according to the implementation provisions in Section 8 of the staff report.

Table 5. Conditional SSOs for Surface Water to Support AWRM Program

Reach	Current Instantaneous Chloride Objective (mg/L)	Conditional Chloride SSO (mg/L)[a]	Averaging Period
6	100	150	12-month~~Annual~~
5	100	150	12-month~~Annual~~
4B	100	117	3-month
4B Critical Conditions	100	130[b]	3-month[c]

a. The conditional SSOs for chloride in the surface water of Reaches 4B, 5, and 6 shall apply and supersede the existing regional water quality objectives of 100 mg/L only when chloride load reductions and/or chloride export projects are in operation by SCVSD according to the implementation provisions in Section 8.

b. The conditional SSO for Reach 4B under critical condition applies, only if the following conditions and implementation requirements are met:

1. Water supply concentrations measured in Castaic Lake are ≥ 80 mg/L.
2. Salt-sensitive agricultural uses that are irrigated with surface water are provided supplemental water during periods when Reach 4B surface water exceeds 117 mg/L.
3. ~~Beginning~~ By May 4, 2020~~16~~, the 10-year cumulative net chloride loading above 117 mg/L ($CNCl_{117}$)[i] to Reach 4B of the SCR, calculated annually, from the SCVSD Water Reclamation Plants (WRPs) ~~is~~ shall be zero or less~~, where:~~.

[i] $CNCl_{117} = Cl_{(Above\ 117)} - Cl_{(Below\ 117)} - Cl_{(Export\ Ews)}$

Where:

$Cl_{(Above\ 117)}$ = [WRP Cl Load[1]/Reach 4B Cl Load[2]] * [Reach 4B Cl $Load_{>117}$[3]]

$Cl_{(Below\ 117)}$ = [WRP Cl Load[1]/Reach 4B Cl Load[2]] * [Reach 4B Cl $Load_{\leq 117}$[4]]

$Cl_{(Export\ EWs)}$ = Cl Load Removed by Extraction Wells

[1] WRP Cl Load is determined as the monthly average Cl concentration multiplied by the monthly average flow measured at the Valencia WRP.

[2] Reach 4B Cl Load is determined as the monthly average Cl concentration at SCVSD Receiving Water Station RF multiplied by the monthly average flow measured at USGS Gauging Station 11109000 (Las Brisas Bridge).

[3] Reach 4B Cl $Load_{>117}$ means the calculated Cl load to Reach 4B when monthly average Cl concentration is above 117 mg/L.

[4] Reach 4B Cl $Load_{<=117}$ means the calculated Cl load to Reach 4B when monthly average Cl concentration is below or equal to 117 mg/L.

4. The chief engineer of the SCVSD signs under penalty of perjury and submits to the Los Angeles Regional Water Quality Control Board (Regional Board) a letter documenting the fulfillment of conditions 1, 2, and 3.

c. The averaging period for the critical condition SSO may be reconsidered based on results of chloride trend monitoring after the alternative water resources management (AWRM) system is applied.

The conditional SSOs for chloride in Reach 4B are applied as 3 month rolling averages because there is salt-sensitive agriculture in the area of Reach 4B and the LRE supplemental study recommended a three-month averaging period for salt-sensitive crops (Newfields, 2008). The conditional SSOs for chloride in Reaches 5 and 6 are applied as 12-month~~annual~~ rolling averages since agriculture in these reaches is identified as non-salt sensitive. ~~Annual~~ Twelve-month averaging periods have been used historically in the Los Angeles Region and throughout California for salts objectives, and an 12-month~~annual~~ average would protect the groundwater recharge and non-salt sensitive agricultural beneficial uses in Reaches 5 and 6 (LWA, 2008).

Table 6. Conditional SSOs for Groundwater to Support AWRM Program

Constituent		Santa Clara--Bouquet & San Francisquito Canyons		Castaic Valley		Lower area east of Piru Creek[1]	
		Conditional SSO (mg/L)	Current Objective (mg/L)	Conditional SSO (mg/L)	Current Objective (mg/L)	Conditional SSO (mg/L)	Current Objective (mg/L)
	Chloride	150	100	150	150	150	200
Averaging period		~~Annual~~12-month	None	12-month	None	~~Annual~~ 12-month	None

[1] Applies only to San Pedro formation. Existing objective of 200 mg/L applies to shallow alluvium layer above San Pedro formation.

The conditional SSOs for chloride in groundwater in Santa Clara-Bouquet & San Francisquito Canyons, Castaic Valley, and the lower area east of Piru Creek (San Pedro formation) shall apply and supersede the existing regional water quality objectives ~~of 100 mg/L~~ only when chloride load reductions and/or chloride export projects are in operation by the SCVSD according to the implementation provisions in Section 8 of the staff report.

4.2. Conditional Wasteload Allocations to Support AWRM

The conditional WLAs for chloride for all point sources shall apply only when chloride load reductions and/or chloride export projects are in operation by the SCVSD according to the implementation provisions in Section 8 of the staff report. If these conditions are not met, WLAs are based on existing water quality objectives for chloride of 100mg/L.

Discharges to Reach 4B by the Saugus and Valencia WRPs will receive the concentration-based conditional wasteload allocations for chloride presented in Table 7.

Table 7. Conditional Reach 4B Wasteload Allocations for chloride for Saugus and Valencia WRPs

Reach	Conditional Chloride SSO (mg/L)[a]	~~Averaging Period~~
4B	117 (3-month Average), 230 (Daily Maximum)	~~3-month~~
4B Critical Conditions	130[a] (3-month Average[b]), 230 (Daily Maximum)	~~3-month~~[b]

a. The Conditional WLA under critical conditions shall apply~~ies~~ only if the following conditions and implementation requirements are met:
   1. Water supply concentrations measured in Castaic Lake are ≥ 80 mg/L.
   2. Salt-sensitive agricultural uses that are irrigated with surface water are provided supplemental water during periods when Reach 4B surface water exceeds 117 mg/L.
   3. ~~Beginning~~ By May 4, ~~2016~~2020, the 10-year cumulative net chloride loading above 117 mg/L ($CNCl_{117}$)[i] to Reach 4B of the SCR, calculated annually, from the Saugus and Valencia WRPs ~~is~~ shall be zero or less~~, where:~~.

[i] $CNCl_{117} = Cl_{(Above\ 117)} - Cl_{(Below\ 117)} - Cl_{(Export\ Ews)}$

Where:

$Cl_{(Above\ 117)}$ = ([WRP Cl Load[1]/Reach 4B Cl Load[2]] * [Reach 4B Cl $Load_{>117}$[3]])
$Cl_{(Below\ 117)}$ = ([WRP Cl Load[1]/Reach 4B Cl Load[2]] * [Reach 4B Cl $Load_{\leq 117}$[4]])
$Cl_{(Export\ EWs)}$ = [Cl Load Removed by Extraction Wells]

[1] WRP Cl Load is determined as the as the monthly average Cl concentration multiplied by the monthly average flow measured at the Valencia WRP.
[2] Reach 4B Cl Load is determined as the monthly average Cl concentration at SCVSD Receiving Water Station RF multiplied by the monthly average flow measured at USGS Gauging Station 11109000 (Las Brisas Bridge).
[3] Reach 4B Cl $Load_{>117}$ means the calculated Cl load to Reach 4B when monthly average Cl concentration is above 117 mg/L.

[4] Reach 4B Cl Load$_{\leq 117}$ means the calculated Cl load to Reach 4B when monthly average Cl concentration is below or equal to 117 mg/L.

4. The chief engineer of the SCVSD signs under penalty of perjury and submits to the Regional Board a letter documenting the fulfillment of conditions 1, 2, and 3.

b. The averaging period for the critical condition WLA may be reconsidered based on results of chloride trend monitoring after the AWRM system is applied.

Beginning May 4, 2015, discharges to Reaches 5 and 6 by the Saugus and Valencia WRPs, will have conditional concentration-based and mass-based WLAs for chloride based on conditional SSOs (Table 8).

Table 8. Conditional WLAs for Saugus and Valencia WRPs

WRP	Concentration-based Conditional WLA for Chloride ~~(12-month Average)~~	Mass-based Conditional WLA for Chloride (12-month Average)
Saugus	150 mg/L (12-month Average), 230 (Daily Maximum)	Q_{Design}*150 mg/L*8.34
Valencia	150 mg/L (12-month Average), 230 (Daily Maximum)	Q_{Design}*150 mg/L*8.34 – AF_{RO}

AF_{RO} is the chloride mass loading adjustment factor for operation of RO facilities, where:

If RO facilities are operated at ≥ 50% ~~rated capacity~~Capacity Factor[a] in preceding 12 months

$$AF_{RO} = 0$$

If RO facilities are operated at < 50% Capacity Factor~~rated capacity~~[b] in preceding 12 months

$$AF_{RO} = (50\% \text{ Capacity Factor } \sout{\text{rated capacity}} - \%\text{RO Capacity}) * \textit{ChlorideLoadRO}^{c}$$

[a] ~~Rated capacity~~Capacity Factor is based on 3 MGD of recycled water treated with RO, 90% of the time.

[b] If operation of RO facilities at <50% ~~rated~~ capacity factor is the result of conditions that are outside the control of SCVSD, then under the discretion of the Executive Officer of the Regional Board, the AF_{RO} may be set to 0.

[c] Chloride load reduction is based on operation of a ~~3 MGD~~ RO treatment plant treating 3 MGD of recycled water with chloride concentration of 50 mg/L + Water Supply Chloride. Assumes operational capacity factor of 90% and RO

membrane chloride rejection rate of 95%. Determination of chloride load based on the following:

$$ChlorideLoadRO = 90\% \times [(Q_{RO} \times C_{WRP} \times 8.34) \times r] \times \left(30 Days / Month \right)$$

where:

Q_{RO} = ~~RO Treatment Flow in MGD (~~3 MGD of recycled water treated with RO~~)~~
C_{WRP} = Chloride Concentration in State Water Project + 50 mg/L
r = % RO chloride rejection (95% or 0.95)
8.34 = Conversion factor (ppd/(mg/L*MGD))

The GSWI model accounted for existing major and minor NPDES dischargers located within the model boundaries. The future modeling scenarios were based on:

- Projected flow for the Saugus and Valencia WRPs and chloride concentrations equal to conditional WLAs,
- projected flow for the Newhall WRP and a chloride concentration of 100 mg/L, and
- existing flow and chloride concentrations for the other major and minor NPDES dischargers.

The affect of assigning conditional WLAs to the Newhall WRP and the other major and minor NPDES discharges on net chloride loading was not modeled. Therefore, other major NPDES dischargers (as defined in Table 4-1 of the Basin Plan), including Newhall WRP, receive WLAs equal to 100 mg/L. The Newhall Ranch WRP already has a permit limit of 100 mg/L for chloride in Order No. R4-2007-0046 based on the current WQO. The Regional Board may consider assigning conditional WLAs for other major NPDES dischargers, including Newhall WRP, based on an analysis of the downstream increase in net chloride loading to surface water and groundwater as a result of implementation of conditional WLAs. The Regional Board may require chloride mass removal quantity that is proportional to mass based chloride removal required for the Valencia WRP in order to receive conditional WLAs.

Other minor NPDES dischargers (as defined in Table 4-1 of the Basin Plan) receive conditional WLAs. Minor discharges receive conditional WLAs without the additional analysis because, based on their flows, the impact of minor discharges is negligible compared to the WRPs.

~~Other NPDES discharges contribute a minor chloride load.~~ The conditional WLAs for minor~~these~~ point sources are presented in Table 9.

Table 9. Conditional WLAs for Minor~~Other~~ NPDES Discharges

Reach	Concentration-based Conditional WLA for Chloride (mg/L)	~~Averaging Period~~
6	150 (12-month Average), 230 (Daily Maximum)	~~Annual~~
5	150 (12-month Average), 230 (Daily Maximum)	~~Annual~~
4B	117 (3-month Average), 230 (Daily Maximum)	~~3-month~~

The WLA of 230 mg/L for daily maximum for chloride is to protect threatened and endangered species. The Endangered Species Protection study indicates that the existing US EPA chronic chloride criteria of 230 mg/L can be considered to be fully protective of local biota.

The final WLAs for TDS and sulfate are equal to existing surface water and groundwater quality objectives for TDS and sulfate in Tables 3-8 and 3-10 of the Basin Plan. The Regional Board may revise the final WLAs based on review of trend monitoring data as detailed in the monitoring section (Section 8.7) of this staff report.

4.3. Conditional Load Allocations to Support AWRM

The source analysis indicates nonpoint sources are not a major source of chloride. The conditional load allocations (LAs) for nonpoint sources are presented in Table 10.

Table 10. Conditional LAs for Nonpoint Sources

Reach	Concentration-based Conditional LA for Chloride (mg/L)	~~Averaging Period~~
6	150 (12-month Average), 230 (Daily Maximum)	~~Annual~~
5	150 (12-month Average), 230 (Daily Maximum)	~~Annual~~
4B	117 (3-month Average), 230 (Daily Maximum)	~~3-month~~

The conditional LAs shall apply only when chloride load reductions and/or chloride export projects are in operation by the SCVSD according to the implementation

provisions in Section 8 of the Staff Report. If these conditions are not met, LAs are based on existing water quality objectives of 100 mg/L.

The LA of 230 mg/L for daily maximum for chloride is to protect threatened and endangered species. The Endangered Species Protection study indicates that the existing US EPA chronic chloride criteria of 230 mg/L can be considered to be fully protective of local biota.

5. Water Code Section 13241 Analysis

In setting site specific objectives, Porter-Cologne section 13241 requires consideration of six factors relating to beneficial uses, economics, the environmental setting, water quality that can be reasonably attained, housing and the need for recycled water. Further, because some of these site specific objectives are greater than the existing water quality objectives, state and federal antidegradation provisions must be considered. These considerations were provided in the Task 7 and 8 Report (LWA, 2008) and are summarized below. Because the agricultural beneficial use of water has been determined to be the most sensitive use under the chloride TMDL, the 13241 analysis focused on salt sensitive agricultural uses. Based on an analysis of the Task 7 and 8 Report, staff concludes that the conditional SSOs, when implemented with the AWRM Program, will support beneficial uses and is in the best interests of the people of California.

5.1. Past, present, and probable future beneficial uses of water

Probable future beneficial uses of the surface waters in Reaches 4, 5, and 6 are likely to remain consistent with past and present uses with the exception of agriculture supply. Agricultural uses in Reaches 5 and 6 will likely decline over time due to increasing urbanization. Agricultural uses in Reaches 4A and 4B will likely remain constant.

The proposed conditional SSOs of 150 mg/L for surface and groundwater within Reaches 5 and 6 are protective of the AGR beneficial use because these waters are not currently and have not historically been used as an irrigation supply for salt-sensitive crops. Newhall Land and Farm is the only landowner with existing agricultural operations that could potentially be impacted by groundwater-surface water interactions within Reach 5 of the Santa Clara River. Newhall has not historically and does not plan in the future to cultivate salt-sensitive crops in Reaches 5 or 6 because of adverse climatic conditions. A number of commercial and wholesale nurseries are located in the Santa Clarita Valley along the Castaic Creek and South Fork tributaries and east of Reach 6, but these nurseries are not likely impacted by surface flows from the Santa Clara River. This situation is unlikely to change due to climatic conditions that impact the ability to grow salt sensitive crops and because the use of irrigation water for crops is anticipated to decline in Reaches 5 and 6 due to planned urban development.

When implemented with the AWRM compliance option, the proposed conditional SSOs of 117 mg/L during normal conditions and 130 mg/L during drought conditions in Reach 4B and the underlying groundwater will protect agricultural uses in the area. Local growers in this area irrigate crops primarily with groundwater from local aquifers fed by releases from Lake Piru and the Santa Clara River, as well as surface diversions from the Santa Clara River. Agricultural supply water originating from Lake Piru are unaffected by chloride levels in the Santa Clara River because Lake Piru is fed with State Water Project water and local runoff. Camulos Ranch is the only known avocado grower that irrigates crops using water originating from Reach 4B waters. The proposed

conditional SSOs in Reach 4B and the underlying groundwater are fully protective of agricultural uses in this area based on the result of the LRE for salt-sensitive crops (a 117 mg/L chloride threshold value) and supplemental water supply to Camulos during drought conditions.

5.2. Environmental characteristics

The environmental setting of the proposed conditional SSOs and TMDL conditional WLA revisions is presented in Section 2.3. The proposed conditional SSOs and TMDL revisions will impact reaches 4B, 5, and 6 of the Santa Clara River and the groundwater basins underlying those reaches. The proposed conditional SSOs, when implemented with the AWRM Program, will ensure protection of beneficial uses considering the environmental characteristics of and the water quality available to the USCR.

Surface flows in the USCR correspond to seasonal precipitation within the region. Portions of the river are perennial, but various reaches are ephemeral and intermittent and flow only during significant storm events. Base flow in the USCR is comprised of surfacing groundwater, discharges from the Saugus and Valencia WRPs, conservation releases of imported and local waters from reservoirs, and agricultural and urban runoff. Base flow in Reach 6 is largely dependent on discharges from the Saugus WRP. Base flows in Reaches 5 and 4B are dependent on Saugus and Valencia WRP discharges as well as rising ground water. Further downstream, in Reach 4A between the confluence at Piru Creek and Las Brisas, surface flow is typically present only during parts of the wet season, which varies by water year. This “dry gap” seasonally separates the upper Santa Clara River hydrologically from the lower river, which, during normal or below normal water years, impedes inter-reach migration and movement of aquatic life. The Vern Freeman Diversion, at the bottom of Reach 3, diverts up to 375 cubic feet per second (cfs) from the Santa Clara River to the El Rio and Saticoy spreading grounds, where the water recharges the underground aquifers and is distributed for agricultural irrigation.

The largest source of chloride to the Upper Santa Clara River is the water supply (see Section 2.5). Dry and critically dry periods affecting the Sacramento and San Joaquin River Valleys reduce fresh-water flow into the Sacramento-San Joaquin Delta and result in higher than normal chloride concentrations in the SWP supply within the California aqueduct system. Typically, water pumped through the Sacramento-San Joaquin Delta takes approximately 1 to 2 years to show up as deliverable SWP water sold by the Santa Clarita Valley wholesaler, CLWA, to local retail water purveyors, due to reservoir storage and turnover time. Salinity fluctuations in the SWP are reflected in both the imported water treated and delivered by the CLWA and the WRP effluent quality. The quality of the SWP water can be high enough to cause or contribute to exceedances of the current water quality objective.

The proposed conditional SSOs are more stringent than historical effluent limitations for the Saugus and Valencia WRPs and would result in improved water quality over existing conditions. In addition, the proposed conditional SSOs are below

the USEPA aquatic life chloride criteria, which according to the TES Study are protective of the most chloride-sensitive organisms for which data are available. Therefore, it is not expected that the proposed conditional SSOs will harm in-stream or riparian species or habitat.

5.3. Water quality conditions that could reasonably be achieved

A detailed discussion of the compliance options and water quality that can be achieved through different approaches to compliance is presented in the Task 2B-1 and Task 2B-2 Reports (Geomatrix, 2008a, CH2MHill 2008, and Geomatrix 2008b). As discussed in Section 5, the AWRM compliance strategy will result in compliance with the proposed conditional SSOs. Other compliance measures, such as large scale advanced treatment facilities, could achieve 100 mg/L in Reaches 5 and 6, but would not meet 100 mg/L during all times in Reach 4B. Given the technical constraints on large scale advanced treatment facilities and the environmental and water resource benefits of the AWRM, staff recommends the adoption of conditional SSOs. Implementation of the AWRM will protect beneficial uses, improve the water quality in the Eastern Piru groundwater basin through export of salts, and result in an overall salt balance in the watershed.

5.4. Economic Considerations

Costs of complying with the existing WQOs were compared with costs of complying with conditional SSOs, including with facility upgrades to the Saugus and Valencia WRPs and other AWRM actions and summarized below.

5.4.1 Compliance with existing WQOs

The costs of two advanced treatment alternatives were analyzed for compliance with existing WQOs. One alternative involves constructing a 3.6 MGD MF/RO facility at the Saugus and WRP and a 15.4 MF/RO facility at the Valencia WRP, so that the entire discharge at each plant meets 100 mg/L in all conditions. This alternative would require brine waste disposal through a pipeline and ocean outfall. A second alternative involves reducing the amount of discharge from each WRP, so that only the minimum amount of discharge necessary to maintain habitat complies with 100 mg/L under all conditions. In this alternative, approximately 6 MGD would be treated with MF/RO at both plants and the remaining balance of effluent would be disposed to a pipeline to the ocean. The estimated capital and operation and maintenance (O&M) costs for these treatment alternatives are in Table 11.

Table 11: Costs for Advanced Treatment to Comply with Existing Objectives

Facility	Capital Cost	Annual O&M
Maximum Advanced Treatment	$118,000,000	$~~8,79~~9,00,000
Brine Disposal	$230,000,000	$~~75~~500,000
Total Maximum Advanced Treatment and Brine Disposal	**348,000,000**	**$9,~~72~~00,000**
Minimum Advanced Treatment	$~~49~~52,000,000	$4,~~42~~00,000
Ocean Discharge	$419,000,000	500,000
Total Minimum Advanced Treatment and Ocean Discharge	**$~~468~~471,000,000**	**$4,~~97~~00,000**

Assuming an interest rate of 5.5% and a period of 20 years, the combined present worth of the estimated Capital and O&M Costs for compliance by providing maximum advanced treatment and brine disposal is approximately $~~460~~ 470 Million and by providing minimum advanced treatment and ocean discharge is $~~524~~ 530 Million. Therefore, the range of costs for facilities required to comply with the existing water quality objectives is between $4~~76~~0 Million and $~~524~~ 530 Million.

5.4.2 Compliance with Conditional SSOs

Cost estimates were prepared for the various elements of the AWRM Program (Table 12). The costs of source control measures are based on SRWS removal and conversion of bleach-based disinfection processes at the WRPs to UV disinfection facilities. The AWRM program also includes construction and operation of a 3-MGD MF/RO facility at the Valencia WRP and brine waste disposal through deep well injection technology. During periods of extreme drought and prior to construction and operation of the MF/RO facility, the AWRM Program includes supplemental water from local water purveyors to reduce chloride levels in the surface water in Reach 4B. Costs for this element were estimated based on a need for approximately 30,000 acre-feet of supplemental water at an assumed cost of approximately $1,000 per acre-feet (based on discussions with local water purveyors) as well as infrastructure for conveyance of the supplemental water at a cost of approximately $7.5 Million. Finally, the costs of water supply facilities needed to achieve salt export from the Piru groundwater basin and blend groundwater with RO permeate include the costs of 10 groundwater extraction wells, a 12-mile RO permeate conveyance pipeline, and a 6-mile blended water supply pipeline.

Table 12. Costs for AWRM Program

AWRM Element	Capital Cost	Present Worth O&M	TOTAL
Source Control Measures	$185,590,000	$6,000,000	$241,950,000
Advanced Treatment and Brine Disposal	$78,400,000	$44,200,000	$122,600,000
Supplemental Water	$37,500,000	N/A	$37,500,000
Ventura Water Supply Facilities	$70,100,000	$3,600,000	$73,700,000
TOTAL AWRM Program	**$2041,1900,000**	**$53,8600,000**	**$2584,700,000**

Note: All costs are as of September 2007

Assuming an interest rate of 5.5% and a period of 20 years, the combined present worth of the Capital and O&M cost for the AWRM facilities required to comply with the proposed site-specific objectives is estimated at approximately $2595 Million.

Amortizing the total costs at 5.5% per year for 20 years yields an annual cost estimate of $36.4005 per month per connection for maximum advanced treatment and brine disposal, $41.5507 for minimum advanced treatment and ocean discharge, and $20.3019.96 for the AWRM. Amortizing the total costs at 5.5% per year for 30 years yields an annual cost estimate of $29.6331.54 per month per connection for maximum advanced treatment and brine disposal, $34.9733.76 for minimum advanced treatment and ocean discharge, and $176.431 for AWRM. This rate analysis does not include additional costs related to procurement of bonds, provision for rate ramp-up periods, nor actual increased costs of project implementation that can occur in the field (e.g., construction change orders, increased cost of materials, and increased cost of construction).

Regional Board staff also reviewed the State Board report, Wastewater User Charge Survey Report F.Y. 2007-2008. This report is prepared annually by the State Board and summarizes and analyzes cost data from a survey of California wastewater agencies. The report shows that the monthly user charge for the City of Santa Clarita was $16.29 per month. The report also shows the statewide monthly service charge average is $33.82 per month and the median is $26.83 per month, with a high of $231.92. For Los Angeles County, the monthly service charge average is $23.90 per month and the median is $12.28 per month. For Ventura County, the monthly service charge average is $38.47 per month and the median is $35.35 per month. The rate will likely increase to a level similar to thenot substantially above the statewide average if applying the AWRM program, and to a level substantially higher than the statewide average if applying the other two options. Potential cost savings to community residents which could be acquired through funding programs to assist in the construction costs, and avoidance of additional treatment costs for other pollutants (i.e. future TMDL requirements) are not included.

5.5. The Need to Develop Housing

The proposed water quality objectives would not restrict the development of housing near the reaches of the Santa Clara River affected by the proposed conditional SSOs because they do not result in discharge requirements that affect housing or housing development. The proposed conditional SSOs and AWRM Program were developed based on projected population and housing growth in the Santa Clarita Valley. The GSWI model considered increased effluent flow from the WRPs and the effects of this growth on the chloride levels in the Santa Clara River and underlying aquifers. The proposed conditional SSOs will support water recycling and the use of the AWRM compliance option in the USCR. Both of these factors will provide water resources to support housing that may be lost with other compliance options.

5.6. The Need to Develop and Use Recycled Water

The proposed water quality objectives will support the expansion of recycled water uses in the Santa Clarita Valley consistent with the California's stated goal of increasing the use of recycled water to help meet the state's growing demand for potable water. The CLWA 2005 Urban Water Management Plan projects that water demand in the area will continue to increase, and that additional sources of water including recycled water will be necessary to meet projected demand. Recycled water use in CLWA's service area is projected to increase from 448 acre-feet per year (actual use in 2004) to 17,400 acre-feet per year by 2030. This 2030 figure represents 70% of the imported water portion of the ultimate wastewater flow projected for the Saugus and Valencia WRPs of approximately 34 MGD. The increased flow from the WRPs from current flows of 21 MGD to future flows of 34 MGD is expected to accommodate most of the increased recycled water demand in the watershed.

The proposed conditional SSOs will support the expansion of recycled water uses by protecting municipal supply. For groundwater recharge reuse projects, Maximum Contaminant Level (MCL) codified in California Administrative Code, Title 22 provide reasonable protection of groundwater quality for the beneficial use of municipal supply. The proposed groundwater objectives for chloride are below the Recommended Secondary Maximum Contaminant Levels for drinking water sources codified in Title 22. Given the demonstrated need to expand recycling in the USCR to meet the region's future water requirements, the proposed conditional SSOs are needed to ensure the required compliance mechanisms allow for the recycling to take place. Additionally, the proposed conditional SSOs are consistent with the secondary MCLs in Title 22 and will not result in water quality for chloride that exceeds these levels.

6. Antidegradation Analysis

State Board Resolution 68-16, “Statement of Policy with Respect to Maintaining High Quality Water” in California, known as the "Antidegradation Policy," protects surface and ground waters from degradation. It states that waters having quality that is better than that established in effective policies shall be maintained unless any change will be consistent with the maximum benefit to the people of the State, will not unreasonably affect present and anticipated beneficial uses, and will not result in water quality less than that prescribed in the policies.

The federal antidegradation policy (40 CFR 131.12) requires states to maintain and protect existing instream water uses and the water quality necessary to protect the existing uses (Tier I), maintain high quality waters unless the State finds after satisfaction of intergovernmental and public participation provisions of the states continuous planning process that allowing lowering water quality is necessary to accommodate important economic and social development (Tier II), and maintain and protect water quality in waters the state has designated as outstanding National resource waters (Tier III).

Adoption of proposed conditional SSOs, when implemented the AWRM Program, would be consistent with the state and federal antidegradation policies. Staff worked with stakeholders to develop a complete antidegradation analysis that is contained in the Task 7 and 8 report (LWA, 2008). The following contains a summary of the antidegradation analysis.

The proposed conditional site specific surface and groundwater objectives are protective of present and anticipated beneficial uses. The proposed conditional SSOs in Reaches 5 and 6 of 150 mg/L are protective of present and anticipated uses for irrigation of non-salt sensitive crops in the area, municipal supply, and aquatic life. The proposed conditional SSOs for Reach 4B, when implemented with the AWRM compliance option, are protective of the present and anticipated beneficial uses of these waters, including the most sensitive beneficial use, salt sensitive agriculture. The proposed SSO of 117 mg/L is within the LRE guidelines for protection of salt sensitive agricultural uses. The proposed SSO of 130 mg/L, which applies during critical conditions when source water is greater than 80 mg/L chloride, is protective when alternative water supplies are provided to salt sensitive agriculture uses (conditional SSO = 130 mg/L) and salt export projects as described in Section 8 are operated such that the net chloride loading above 117 mg/L is zero or less.

The proposed implementation activities, which will increase chloride export from the East Piru groundwater basin, will offset any increases in chloride discharges. If higher water quality objectives (130 mg/L) are in place in Reach 4B due to elevated concentrations in source water, the groundwater basin will be protected from degradation through the required salt export. The AWRM proposal will improve water quality in the basin over time and offset any increase in chloride concentrations that result from the

higher objective during some periods. The AWRM proposal was evaluated based on design capacities at Valencia WRP and Saugus WRP of 27.6 MGD and 6.5 MGD, for a total system design capacity of 34.1 MGD. If the capacity of the WRPs ever exceeds the current total system design capacity of 34.1 MGD, then the amount of water required for salt reduction and/or export should increase proportionally to the increase in the total system design capacity, and an additional antidegradation analysis should be conducted.

Under the AWRM Program, the blended extraction well and RO permeate discharge into Reach 4A will not exceed a chloride concentration of 95 mg/L. The current chloride WQO of 100 mg/L in Reaches 3 and 4A is within the LRE guidelines and will protect salt-sensitive agricultural uses. Therefore, the blended extraction well and RO permeate discharge into Reach 4A will not exceed the WQO of the receiving water at the point of discharge (Reach 4A) or in the reach downstream of the discharge point (Reach 3) and the designated beneficial uses for the reaches are still protected. This satisfies EPA's Tier 1 requirements in 40 CFR 131.12(a). Ongoing trend monitoring and additional modeling will determine whether the blended extraction well and RO permeate discharge would increase chloride concentrations in high quality waters downstream in Reaches 4A and 3 and in the Fillmore and Santa Paula groundwater basins. The GSWI model will be extended to the Freeman Diversion to assess the interaction of groundwater and surface water through the Piru, Fillmore, and Santa Paula groundwater basins and the overlying surface waters.

The proposed conditional SSOs and implementation of the AWRM are consistent with the maximum benefit to the people of the state and will result in social and economic benefits. It has been shown that AWRM Program will support water recycling and provide for additional water resources for agriculture and aquatic habitat. The GSWI model demonstrates that the AWRM compliance option results in benefits from the County Line to the area of seawater intrusion on the Oxnard Plain. The model shows that the ARWM option allows for more water diverted at the Freeman Diversion than conventional advanced treatment options, which then has a significant effect on saline intrusion in the Oxnard Plain. At the downstream end of the Piru basin, modeled surface water chloride concentrations are higher in the river about 40% of the time with the AWRM operating, but still in compliance with the existing water quality objective of 100 mg/L. Groundwater chloride concentrations in Piru Basin are improved by pumping and replacing groundwater with stormwater recharge during wet years when chloride concentrations are lower. As a result, surfacing groundwater from the Piru basin in Reach 4A may decrease over time as a result of the AWRM. The AWRM will also result in increased surface water flows in Reaches 3 and 4A as compared to other compliance options. Additionally, the proposed groundwater and surface water objectives for Reaches 5 and 6 will support the expansion of recycled water uses in the Santa Clarita Valley, which is consistent with the maximum public benefit and not unreasonably adverse to present and anticipated beneficial uses. Finally, in general, the AWRM compliance option has more water quality benefits to Ventura County than do the conventional advanced treatment based compliance options.

The proposed conditional SSOs will not result in water quality less than that prescribed in the policies. The proposed conditional SSOs comport with the Chloride Policy in Regional Board resolution 97-002 and its requirements for a watershed chloride reduction plan.

Finally, the proposed conditional SSOs will be implemented through NPDES permits, including effluent limits and required minimum salt export requirements. The effluent limits will ensure that the current performance of the WRPs continues at a minimum and will most likely require additional actions to achieve the water quality objectives. Additionally, receiving water limits will ensure that downstream water quality is not degraded as a result of wastes discharged. Finally, minimum salt export requirements will be included to ensure that excess salt loadings to the groundwater basin due to periods of elevated water supply concentrations are removed from the groundwater basin through pumping and export.

7. Alternatives Analysis and Staff Recommendation

Based on the results of the TMDL special studies, Regional Board staff analyzed two alternatives for Regional Board consideration. The first entails a TMDL based on the existing surface water Basin Plan objectives; the second alternative entails a TMDL based on a suite of site specific objectives for both surface water and groundwater underlying the Upper Santa Clara River to support the AWRM approach. Both alternatives rely on implementation of RO technology; however, the first alternative requires larger capacity RO facilities and ocean brine disposal while the second alternative requires smaller capacity RO facilities and no ocean disposal.

7.1. Alternative 1 - Maintain Current Basin Plan Objectives – No Action

Under this alternative, the Regional Board takes no action at this time to adopt SSOs or amend the TMDL Wasteload Allocations and Implementation Schedule. Staff notes several concerns with Alternative 1.

First, a key factor in implementation of RO is safe disposal of the resultant brine waste. Several options for brine disposal include ocean discharge, deep well injection, and drying and subsequent landfill disposal. Cost-effective brine disposal is based on several factors including the brine quantity generated and proximity to available disposal facilities. Because it requires larger capacity RO to meet more stringent objectives, the first alternative would require brine disposal via an ocean discharge. The second alternative, which requires smaller capacity RO, would enable disposal via deep well injection. Ocean disposal options generally provide greater capacity than disposal wells, but for the Santa Clarita Valley, would require construction of a large pipeline through two counties over 43-miles. Deep well injection involves retrofitting abandoned oil production wells or constructing new injection wells in areas near the Santa Clarita Valley and injecting the brine into stable geological formations. Local disposal of the smaller volumes brine associated with second alternative through deep well injection or landfilling is likely more cost effective and would likely have less environmental impacts than ocean disposal for this site. In particular, facilities for deep well injection are closer to the RO facilities than ocean disposal sites and therefore require a shorter pipeline. Further, the capacity limits the size of the RO plant so that electrical resources are lower than the first option.

Another concern with the first alternative is under an ocean disposal scenario, a pipeline and outfall could potentially be used for discharge of treated wastewater rather than the discharge of brine. If the SCVSD were to discharge wastewater directly to the Ocean, this option would reduce flows in the Upper Santa Clara River.

7.2. Alternative 2 - Adopt Conditional SSOs and Revised TMDL Conditional WLAs

Under this alternative, the Regional Board adopts a suite of site specific objectives that are conditioned on implementing a chloride balance that is based on advanced treatment of the Valencia WRP effluent to reduce chloride loading to the USCR by a level greater than any loading contributed by the Valencia WRP in excess of loading corresponding to 117 mg/L (see section 8.2). TMDL conditional WLAs for chloride are revised to reflect the conditional SSOs. In addition, interim WLAs for sulfate and TDS are included to facilitate the use of supplemental water to Reach 4B when chloride objectives exceed 117 mg/L.

The AWRM Program uses smaller-scale reverse osmosis to provide greater flexibility for disposal of brine generated by the reverse osmosis system. The AWRM Program also provides capability for aquifer restoration and resource conservation through blending the advanced treated wastewater with extracted groundwater from degraded underlying basin in the upper Santa Clara River. In order to implement an alternative implementation plan, conditional SSOs that are in excess of the existing WQOs for surface water are required. However, because the AWRM facilitates the feasibility of aquifer restoration, the groundwater WQOs can be more stringent. This alternative is analyzed in accordance with a salt balance in the Upper Santa Clara River Watershed.

7.3. Staff Recommendation

Staff recommends the adoption of Alternative 2- adopt conditional site specific objectives and revised TMDL conditional WLAs. The conditional site specific objectives will maintain beneficial uses and the implementation of the AWRM program will result in decreased salt loading to the USCR with fewer environmental and economic impacts than Alternative 1. Additional benefits in both water supply and water quality accrue in areas downstream of the USCR.

- Staff finds that the key technical issues of cumulative chloride impacts to groundwater have been addressed by GSWI. Details of staff's findings on the GSWI model are presented in Appendix I, "GSWI Study for the USCR Chloride TMDL – Staff Report."

- Staff find that the GSWI model has been adequately calibrated by 88 groundwater level, 50 groundwater chloride, 6 streamflow, and 12 surface-water quality target locations that are spatially distributed throughout the GSWI domain and it has been considered as an appropriate model for groundwater and surface water interaction modeling purposes.

- Staff finds that, based on the GWSI model, none of the simulated chloride concentrations derived from the proposed compliance options result in chloride concentrations less than the existing WQO of 100 mg/L in surface water at all

times over 24-year simulation periods (2007-2030) and at all locations in Reaches 4B, 5 and 6. All of the predicted chloride concentrations in groundwater for all compliance options consistently met the existing WQO of 200 mg/L in groundwater of the Piru Basin except the area between Blue Cut and SCR-RF monitoring locations.

- Staff finds that the model predicted high chloride concentrations of 350 mg/L or greater in the alluvial groundwater (thickness of 50-100 ft) in the areas between Blue Cut and receiving water station SCR-RF during drought periods for all proposed compliance options. The high chloride concentration in this area will migrate downstream through the pumping activity in the proposed extraction well locations for the AWRM compliance option and will affect the chloride concentration of the mixed water with RO and then will affect the chloride concentration in SCR in Reach 4A. Geomatrix has prepared a technical memo stating that there is no current or expected future use of the shallow groundwater for beneficial uses in this area (Geomatrix, 2008c). The memo states that groundwater production in Reach 4B for existing beneficial uses occurs downstream of Blue Cut area, where the aquifer has a greater saturated thickness, yields more water, and has lower chloride concentrations. The memo also states that the alluvial groundwater concentrations are predicted to quickly recover once the drought period has ended. Staff therefore recommends that the proposed SSOs of 150 mg/L be set for the deeper San Pedro Formation and that the existing WQOs of 200 mg/L be retained for the shallow alluvium layer.

- Staff finds that the predicted chloride concentrations in both groundwater and surface water at Blue Cut were generally related to concentrations of chloride in the discharges to the SCR from the Saugus and Valencia WRPs.

- Staff finds that the Advanced Treatment and Brine Disposal Compliance Option can not result in full attainment of the 100 mg/L WQO for the USCR at Blue Cut at all times and in all locations of the receiving water. In addition, other compliance options like conveying all recycled water discharges from the Valencia and Saugus WRPs to the ocean outfall (Zero Discharge Compliance Option), limiting discharges from the WRPs and conveying the balance of WRPs recycled water discharges to ocean outfall (Minimal Discharge Compliance Option), and moving the discharge location of WRPs to the beginning of Reach 7 near Lang gauge (Alternative WRP Discharge Location Compliance Option) are also not likely to achieve attainment of the existing 100 mg/L WQO at all times and all locations.

- Staff notes that an alternative compliance option is required to achieve the site specific objectives (SSOs) when the original proposed compliance options were not able to achieve the existing WQO of 100 mg/L. Staff also notes that the SSOs shall be carefully evaluated based on the GSWI model results of different averaging periods to ensure they are fully protective of the agricultural beneficial uses in the study area.

- Staff finds that the AWRM compliance option can produce better chloride concentrations than other proposed compliance options during drought periods and the salt export capability of the AWRM compliance option will help to substantially reduce the amount of chloride loading from salt-water intrusion in the Oxnard Plain.

- Staff finds that the AWRM compliance alternative will result in timely attainment of conditional SSOs and reduce the chloride load to the USCR and underlying groundwater basins during the TMDL implementation period. Staff further finds that the AWRM will help provide enough mass loading to protect the SCR downstream from sea water intrusion.

- Staff finds that the proposed conditional SSOs would be consistent with state and federal antidegradation policies. The antidegradation analysis shows that the Alternative Water Resources Management Plan, involving conditional SSOs that are less stringent than existing WQOs used in conjunction with advanced treatment and salt export, are protective of beneficial uses in the USCR.

- Staff finds that the proposed conditional SSOs considered section 13241 requirements including: (a) past, present, and probable future beneficial uses of water, (b) environmental characteristics of the hydrographic unit under consideration, (c) water quality conditions that could reasonably be achieved, (d) economic considerations, (e) the need for developing housing within the Region, and (f) the need to develop and use recycled water.

- Staff finds that the AWRM Program is consistent with the draft State Board Water Recycling Policy. A stakeholder draft of the policy was presented to the State Board on September 3, 2008. This draft policy states that salts from all sources should be managed on a basin-wide or sub basin-wide basis to attain water quality objectives and support beneficial uses through the development of regional salt management plans. The draft policy provides some specific requirements to be met in the salt management plans, including:

    1. Basin or sub basin-wide monitoring;

    2. Determination of all sources and loading of salts, the basin's assimilative capacity of salts, and fate and transport of salts;

    3. Implementation measures to manage salt loading on a sustainable basis;

    4. An antidegradation analysis demonstrating that projects included with the plan will satisfy State Board Resolution 68-16; and

    5. Water recycling and stormwater recharge/reuse goals and objectives.

Although no salt management plan has yet been developed for the Santa Clara River watershed, the AWRM program can serve as a basis for a future salt management plan. The AWRM Program elements have many similarities to the required salinity management plan elements. The AWRM Program was developed using the G~~W~~SWI model. Based on the total system design capacity of 34.1 MGD for the Saugus and Valencia WRPs and accommodated future growth, the GSWI model ~~, which~~ assessed the fate and transport of chloride from all sources in the surface waters and groundwater in the Santa Clara River watershed. The GSWI model also assessed water quality impacts associated with the planned recycled water uses in the future. Given that the AWRM program will eventually be implemented through various NPDES permits issued in the future, it also will involve a number of monitoring requirements to assess actual fate and transport of chloride during and after project implementation. While the GSWIM was developed specifically to assess the fate and transport of chloride, the evaluations and assessments will largely apply to other salts in the region, which behave similarly to chloride. The facilities that will be implemented through the AWRM (i.e., advanced treatment of wastewater, salt export facilities) will also remove and manage other salts. Hence, with some minor modifications and assessments, the AWRM program could be deemed a salinity management plan for the watershed, since it would provide for (1) watershed-wide monitoring, (2) determination of all sources, loading, fate and transport of salts, (3) salt management measures and implementation, (4) an antidegradation analysis; and (5) water recycling goals and objectives.

8. Implementation

The conditional SSOs proposed in Section 4.1 are conditioned on implementation of the AWRM program; if the AWRM system is not built, the water quality objectives revert back to the current levels in the Basin Plan (100 mg/L). These conditions comport with the Chloride Policy in Regional Board resolution 97-002 and its requirements for a watershed chloride reduction plan. The watershed chloride reduction plan will be implemented through NPDES permits for the Valencia WRP and a new NPDES permit for discharge into Reach 4A. The conditional site specific objectives for chloride in the USCR watershed shall apply and supersede the regional water quality objectives only when chloride load reductions and/or chloride export projects are in operation and reduce chloride loading in accordance with Table 13.

Table 13. Watershed Chloride Reduction Plan

Water Supply Chloride[1]	Chloride Load Reductions[2]
40 mg/L	58,000 lbs per month
50 mg/L	64,000 lbs per month
60 mg/L	71,000 lbs per month
70 mg/L	77,000 lbs per month
80 mg/L	83,000 lbs per month
90 mg/L	90,000 lbs per month
100 mg/L	96,000 lbs per month

[1] Based on measured chloride of the SWP water stored in Castaic Lake

[2] Chloride load reduction is based on operation of a ~~3 MGD~~ RO treatment plant treating 3 MGD of recycled water with chloride concentration of 50 mg/L + Water Supply Chloride. Assumes operational capacity factor of 90% and RO membrane chloride rejection rate of 95%. Determination of chloride load based on the following:

$$ChlorideLoad = 90\% \times [(Q_{RO} \times C_{WRP} \times 8.34) \times r] \times \left(30 Days / Month\right)$$

where r = % chloride rejection (95%)
Q_{RO} = 3 MGD of recycled water treated with RO~~RO treatment flow (3 MGD)~~
C_{WRP} = SWP Cl + 50 mg/L

8.1. Implementation of Reach 4B Conditional WLAs

The Saugus and Valencia WRP NPDES permits will have receiving water limits for the District's receiving water station, RF, located in Reach 4B of the Santa Clara River. The receiving water limits will be based on the Reach 4B conditional WLAs for chloride as presented in section 4.2.

8.2. Implementation of Reach 5 and 6 Conditional WLAs

Beginning May 4, 2015, Reach 5 and 6 conditional WLAs for the Saugus and Valencia WRPs (Table 5) will become effective. Prior to May 4, 2015, Saugus and Valencia WRPs will have interim WLAs for chloride equal to ~~the interim limits for chloride specified in order Nos. R4-2003-0143 and R4-2003-0145 as amended by order Nos. R4-2005-0031 and R4-2005-0032 (Table 14).~~ the interim limit for chloride specified in order No. R4-04-004.

Table 14. Interim WLAs for Valencia and Saugus WRPs

Reach	Interim Chloride WLA (mg/L)	Interim Sulfate WLA (mg/L)	Interim TDS WLA (mg/L)	Averaging Period
5	[SWP] + 114 not to exceed 230	450	1000	12-month~~Annual~~
6	[SWP] + 134 not to exceed 230	450	1000	12-month~~Annual~~

In addition, in order to support water recycling in the USCR, which is critical to the success of and stakeholder support for the AWRM Program, the Saugus and Valencia WRPs will receive interim WLAs for sulfate and TDS (Table 14). When the water reclamation requirements for these WRPs are renewed, they will likely contain limits based on groundwater WQOs. Current levels of sulfate and TDS in the WRP effluent will not meet limits based on existing WQOs. Instead the Saugus and Valencia WRPs must meet interim WLAs equal to 450 mg/L sulfate and 1000 mg/L TDS, which will apply for discharges to the Santa Clara River and recycled water uses from the Saugus Valencia WRPs. This will allow the SCVSD time to conduct special studies on the impacts of sulfate and TDS concentrations at these levels on groundwater quality and the potential for sulfate and TDS SSOs. These interim WLAs will expire on May 4, 2015 and will be replaced either with final WLAs based on the results of SSOs, if developed, or existing WQOs.

The interim WLAs are protective of beneficial uses and consistent with historical surface and groundwater objectives for basins underlying Reaches 5 and 6. A recent report prepared for the SCVSD used a weight of evidence approach to demonstrate that the interim WLAs for sulfate are protective of USCR aquatic life uses, including threatened and endangered fish and amphibians, and their prey organisms (Environ, 2008). The report states that the species mean acute value of the most acutely sulfate-sensitive invertebrate species was more than four times greater than the interim WLA of 450 mg/L. The report also states that the available toxicity data for sulfate confirm the relatively low sensitivity of fish, including threatened and endangered species in the USCR, to sulfate. Thus, protective values based on highly sensitive invertebrates will be additionally protective of TES fish and amphibians given their low sensitivity to ions.

Additionally, the interim WLAs are protective of groundwater recharge uses. These levels are consistent with the upper range of the secondary MCLs in Title 22.

8.3. Blended RO and Groundwater Discharge to Reach 4A

An NPDES permit and associated Monitoring and Reporting Program (MRP) will be required for any new discharge of the blend of RO-treated recycled water and extracted groundwater from the east Piru Basin, as contemplated in the AWRM Program. The Permittee shall submit a report of waste discharge and initiate an application to receive an NPDES permit for these facilities prior to their discharge to the SCR. Permit writers will consider ambient water quality when establishing permit limits to meet WQOs for Reach 4A.

8.4. Supplemental Water

Supplemental water released to Reach 6 of the Santa Clara River will require an NPDES permit. The AWRM contemplates the use of existing Saugus aquifer wells to deliver low chloride supplemental water directly to the USCR because infrastructure already exists and would not need to be constructed. These supplemental waters would be delivered through contractual arrangements between the SCVSD and the Upper Basin Water Purveyors and would be discharged directly to the USCR. However, although chloride concentrations in these alternative supplemental water wells are very low (20 to 42 mg/L), sulfate concentrations consistently exceed the existing surface water quality objective of 300 mg/L for Reach 6 and the TDS groundwater objectives of 700 mg/L for the groundwater basin underlying Reach 6.

Interim wasteload allocations (Table 12) are developed for sulfate and TDS for the dilution water discharges. These wasteload allocations would apply until then end of the TMDL Implementation period in order to allow (1) time for construction of infrastructure to connect the supplemental water to the Valencia WRP and be diluted with the RO permeate, or (2) time for the SCVSD to conduct additional special studies to provide adequate justification for SSOs for sulfate and TDS. If infrastructure to remove the direct discharge of supplemental water to the USCR is not constructed or if the Regional Board does not approve SSOs for sulfate and TDS, the interim WLAs would expire.

Table 12. Interim WLAs for Reach 6 Supplemental Water Discharges

Reach	Interim Sulfate WLA (mg/L)	Interim TDS WLA (mg/L)	Averaging Period
6	450	1000	12-

	month~~Annual~~

The interim WLAs are protective of beneficial uses and consistent with historical surface and groundwater objectives for Reach 6 (see discussion in section 8.2).

The final WLAs for TDS and sulfate are equal to existing surface water and groundwater quality objectives for TDS and sulfate in Tables 3-8 and 3-10 of the Basin Plan. The Regional Board may revise the final WLAs based on review of trend monitoring data as detailed in the monitoring section (Section 8.7) of this staff report.

8.5. Downstream Effects of TMDL Implementation

Implementation of the USCR Chloride TMDL, including implementation of AWRM and the discharge to Reach 4A of the blended RO permeate and pumped groundwater will not cause exceedances of surface water quality objectives for downstream reaches. The water discharged to Reach 4A will meet the WQO of 100 mg/L for Reaches 4A and 3. Furthermore, US EPA has established a TMDL for chloride in Reach 3 of the Santa Clara River (US EPA, 2003). The TMDL for Reach 3 sets a numeric target of 80 mg/L of chloride. The linkage analysis for the Reach 3 TMDL demonstrates that the numeric target of 80 mg/L will be attained if upstream discharges from Reach 4 have a chloride concentration of 100 mg/L.

Although the discharge to Reach 4A will have a concentration below the surface WQO of 100 mg/L, it will have a concentration greater than the existing chloride concentrations in Reach 4A and the Fillmore groundwater basin downstream. The average chloride concentration in Reach 4A is 59 mg/L, based on data collected from 1992 to 2006 downstream of the Fillmore Fish Hatchery. The GWSI model was used to calculate the average mass loading, average chloride concentration, and average flow from the discharge to 4A of blended RO permeate and extracted groundwater. This was compared with historic chloride concentration and flow data to determine the incremental increase in Reach 4A surface water chloride concentrations caused by the blended discharge. Depending on the flows and existing surface chloride concentrations, the discharge could increase chloride concentrations by up to 20 mg/L in Reach 4A

The increased concentrations in surface water could impact groundwater quality in the Fillmore Basin, depending on how much surface water recharges the groundwater. The average chloride concentration in the Fillmore Basin is 49 mg/L, 62 mg/L, and 46 mg/L based on data collected at wells V-0309, V-0340, and V-0342, respectively, located in the eastern portion of the Fillmore Basin from 1987 to 2006. Therefore, there is a potential to degrade water quality below existing ambient conditions in groundwater by implementation of the AWRM compliance option. The extent of this potential degradation needs to be further assessed through an evaluation of hydrology and the amount of surface water recharge that occurs in Reach 4A and downstream.

In addition, the potential increases in chloride concentrations in the Fillmore Basin, which is the water supply for the City of Fillmore, could impact the levels of chloride in Fillmore treatment plant effluent discharged to Reach 3.

Therefore, it is likely that an antidegradation analysis will be required during the permitting stage for the discharge to Reach 4A. The permit will require further evaluation of this discharge and any impacts on downstream uses, groundwater and surface water monitoring, and enforceable effluent limits. An initial antidegradation analysis is presented here. State and federal antidegradation requirements include the following conditions:

- The reduction in water quality will not unreasonably affect actual or potential beneficial uses.
- The proposed action is necessary to accommodate important economic or social development in the area.
- The reduction in water quality is consistent with maximum public benefit.
- Water quality will not increase above water quality objectives prescribed in the Basin Plan.

The current chloride WQO of 100 mg/L in Reaches 3 and 4A will protect the most sensitive beneficial use of the river's water, which is salt-sensitive agricultural use and has threshold value of 117 mg/L. Under the AWRM Program, the blended extraction well and RO permeate discharge into Reach 4A will not exceed a chloride concentration of 95 mg/L, and may be further adjusted downward as needed to protect water quality. Therefore, the blended extraction well and RO permeate discharge into Reach 4A will not exceed the water quality objective of the receiving water at the point of discharge or in the reach downstream of the discharge point.

Further water quality assessments will be used to determine whether the discharge to 4A would increase chloride concentrations in groundwater in the Fillmore and Santa Paula Basins. Responsible parties, including SCVSD and the ultimate permit holder for the 4A discharge, will be required to conduct chloride trend monitoring in the Fillmore Basin and in Reaches 3, 4A to evaluate impacts of compliance measures to downstream groundwater and surface water quality, including areas downstream of the Fillmore treatment plant. This TMDL shall be reconsidered if chloride trend monitoring indicates degradation of groundwater or surface water due to implementation of compliance measures.

The water quality analyses discussed above will be utilized in conjunction with an extension of the GSWI model to assess the interaction of groundwater and surface water and any potential impacts to downstream water quality by the AWRM option. Specifically, key stakeholders have agreed through a memorandum of understanding to extend the GSWI model through the Piru, Fillmore, and Santa Paula groundwater basins and the overlying surface waters to the Freeman Diversion. If the extended GSWI model results indicate the blended extraction well and RO permeate discharge as currently proposed by the AWRM option would cause an exceedance of water quality objectives,

the GSWIM will be utilized to determine the level of chloride in the blended extraction well and RO permeate discharge necessary to preclude such an exceedance.

The important social and economic benefits of the AWRM Program could warrant some degradation of the downstream reaches. It has been shown that AWRM Program will support water recycling and provide for additional water resources for agriculture and aquatic habitat. Additionally, chloride concentrations in the Santa Clara River will be lower at the Ventura-Los Angeles County Line, and will result in better-quality recharge to the east Piru basin. As a result, surfacing groundwater from the Piru basin in Reach 4A may decrease over time as a result of the AWRM. The AWRM will also result in increased surface water flows in Reaches 3 and 4A as compared to other compliance options. Finally, in general, the AWRM compliance option has more water quality benefits to Ventura County than do the conventional advanced treatment based compliance options.

It is important to note that any degradation in water quality can be averted by operating the extraction wells in the Piru basin in a manner that will not cause increases in the baseline water quality for the Fillmore and Santa Paula groundwater basins and surface water reaches (4A and 3). For example, the maximum concentration of the extraction well and RO permeate blend could be adjusted downward from 95 mg/L, as warranted based on GSWIM modeling.

The Reach 3 Chloride TMDL may be re-evaluated in the context of the findings of the Upper Santa Clara River Chloride TMDL studies, chloride trend monitoring, and the extended GSWI model results.

8.6. Implementation Schedule

The TMDL provides a ten-year schedule to attain compliance with the conditional SSOs and conditional wasteload allocations. Key uncertainties at this point relate to identification of the optimum method for brine disposal. Several options, including deep-well injection in the vicinity of old oil fields in the Santa Clarita Valley, and drying and landfill disposal will be considered by the SCVSD in the first two years of the TMDL Implementation Plan.

The Implementation schedule includes 6 years for implementation of compliance measures including planning, completing Environmental Impact Report, engineering design, and construction. The Regional Board will re-valuate the schedule to implement control measures needed to meet final conditional WLAs at year 6 (2011) and year 9.5 (2014) after the effective date of the TMDL.

8.7. Monitoring for the AWRM Program

NPDES Permittee will conduct TDS, chloride, and sulfate monitoring to ensure that water quality objectives are being met. This monitoring will be consistent with and at least equivalent to monitoring specified in existing permits.

The SCVSD will submit a monitoring plan to conduct chloride, TDS, and sulfate trend monitoring to ensure that the goal of chloride export in the watershed is being achieved, water quality objectives are being met, and downstream groundwater and surface water quality is not degraded due to implementation of compliance measures. The SCVSD monitoring plan shall include plans to monitor chloride, TDS, and sulfate in groundwater and identify representative wells to be approved by the Regional Board Executive Officer, in the following locations: (a) Shallow alluvium layer in east Piru Basin, (b) San Pedro Formation in east Piru Basin, and (c) groundwater basins under Reaches 5 and 6, which shall be equivalent or greater than existing groundwater monitoring required by NPDES permits for Saugus and Valencia WRPs. The monitoring plan shall also include a plan for chloride, TDS, and sulfate trend monitoring for surface water for Reaches 4B, 5 and 6. The monitoring plan shall include plans to monitor chloride, TDS, and sulfate at a minimum of once per quarter for groundwater and at a minimum of once per month for surface water. The plan should propose a monitoring schedule that extends beyond the completion date of this TMDL to evaluate impacts of compliance measures to downstream groundwater and surface water quality. This TMDL shall be reconsidered if chloride, TDS, and sulfate trend monitoring indicates degradation of groundwater or surface water due to implementation of compliance measures.

The Reach 4A permittee will submit a monitoring plan to conduct chloride, TDS, and sulfate trend monitoring to ensure that the goal of chloride export in the watershed is being achieved, water quality objectives are being met, and downstream groundwater and surface water quality is not degraded due to implementation of compliance measures. The Reach 4A permittee monitoring plan shall include plans to monitor chloride, TDS, and sulfate in groundwater and identify representative wells to be approved by the Regional Board Executive Officer in the following locations (a) Fillmore Basin, and (b) Santa Paula Basin. The monitoring plan shall also include a plan for chloride, TDS, and sulfate trend monitoring for surface water for Reaches 3 and 4A. The monitoring plan should include plans to monitor chloride, TDS, and sulfate at a minimum of once per quarter for groundwater and at a minimum of once per month for surface water. The plan should propose a monitoring schedule that shall extend beyond the completion date of this TMDL to evaluate impacts of compliance measures to downstream groundwater and surface water quality. This TMDL shall be reconsidered if chloride, TDS, and sulfate trend monitoring indicates degradation of groundwater or surface water due to implementation of compliance measures. ~~The SCVSD and Reach 4A Permittee will conduct chloride, sulfate, and TDS trend monitoring to ensure that the goal of chloride export in the watershed is being achieved, water quality objectives are being met, and downstream groundwater and surface water quality is not degraded due to implementation of compliance measures. Trend monitoring for groundwater shall be conducted by the SCVSD at the following locations measured at representative wells as determined by the Regional Board Executive Officer: (a) Shallow alluvium layer in east Piru Basin, (b) San Pedro Formation in east Piru Basin, and (c) groundwater basins under~~

~~Reaches 5 and 6, which shall be equivalent or greater than existing groundwater monitoring required by NPDES permits for Saugus and Valencia WRPs. Trend monitoring for groundwater shall be conducted by the Reach 4A Permittee at the following locations measured at representative wells as determined by the Regional Board Executive Officer: (a) Fillmore Basin, and (b) Santa Paula Basin. Chloride trend monitoring for surface water shall be conducted by the SCVSD for Reaches 4B, 5 and 6, while trend monitoring for surface water shall be conducted by the Reach 4A Permittee for Reaches 3 and 4A. Trend monitoring shall be conducted at a minimum of once per quarter for groundwater and at a minimum of once per month for surface water. Trend monitoring shall extend beyond the completion date of this TMDL to evaluate impacts of compliance measures to downstream groundwater and surface water quality. A monitoring plan shall be submitted by the SCVSD and Reach 4A Permittee to the Regional Board for Executive Officer approval within six months after the completion date of Task 10. Monitoring will begin one year after Executive Officer approval of the monitoring plan to allow time for the installation of any monitoring wells and/or surface water monitoring stations. Trend monitoring in Fillmore and Santa Paula Basins and in Reaches 3 and 4A will begin one year after Executive Officer approval of the monitoring plan and upon issuance of NPDES permit for the Reach 4A Permitee. This TMDL shall be reconsidered if chloride trend monitoring indicates degradation of groundwater or surface water due to implementation of compliance measures.~~

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Interconnection Agreement Between County Sanitation Districts Nos. 26 and 32 of Los Angeles County and the Newhall Land and Farming Company

CSD Contr#3868

AGREEMENT

BETWEEN COUNTY SANITATION DISTRICTS NOS. 26 AND 32 OF LOS ANGELES COUNTY AND THE NEWHALL LAND AND FARMING COMPANY REGARDING A PLAN FOR THE COORDINATION OF WASTEWATER MANAGEMENT FACILITIES RELATING TO THE NEWHALL RANCH SPECIFIC PLAN DEVELOPMENT

This Agreement is made and entered into this 9th day of January, 2002, by and between **County Sanitation Districts Nos. 26 and 32 of Los Angeles County** (collectively **Districts** and individually **District No. 26** or **District No. 32**, respectively), special districts organized and existing pursuant to the County Sanitation District Act, California Health and Safety Code Sections 4700 et seq., and **The Newhall Land and Farming Company (NLFC)**, a California limited partnership.

RECITALS

1. NLFC owns land in unincorporated Los Angeles County adjacent to and west of the Districts and is proposing to construct a new development pursuant to the Newhall Ranch Specific Plan.

2. NLFC is currently planning, designing, and engineering the wastewater conveyance, treatment and disposal systems to serve the Newhall Ranch Specific Plan development.

3. A site for a new wastewater treatment plant within the proposed development area is identified in the Newhall Ranch Specific Plan.

4. NLFC has proposed the formation of a new county sanitation district to provide wastewater management services for the Newhall Ranch Specific Plan area pursuant to the County Sanitation District Act of 1923 and the Cortese-Knox-Hertzberg Reorganization Act of 2000.

5. The Districts currently provide wastewater management services for areas immediately adjacent to the Newhall Ranch Specific Plan area.

6. The Districts, together with all of the other County Sanitation Districts of Los Angeles County, are signatory to the "Amended Joint Administration Agreement," effective July 1, 1980, (JAA) providing for the joint management, administration and staffing of the signatory districts.

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7. The Districts are signatory to the "Joint Powers Agreement for a Regional Sewerage System in the Santa Clarita Valley," effective May 8, 1984, (SCVJSSA), which provides for the joint operation of the wastewater management facilities within the Districts.

8. There are benefits in coordinating the wastewater management services between the Districts and the areas that would be served by a new county sanitation district. To this end, NLFC and the Districts have previously entered into a Reimbursement Agreement to cover the expenses for joint planning of wastewater management facilities.

9. It is desirable to prepare a plan for coordinated wastewater management facilities prior to the formation of the new county sanitation district. This plan will serve as a "road map" for the formation of the new county sanitation district and its subsequent operation.

10. The anticipated total flow originating from within the boundaries of the new county sanitation district is 6.9 million gallons per day (mgd).

NOW, THEREFORE, in consideration of the promises and mutual covenants herein contained, the parties hereto agree as follows:

I. FORMATION AND GOVERNING AGREEMENTS

A. NLFC shall request that the Los Angeles County Board of Supervisors petition the Local Agency Formation Commission (LAFCO) to:

1. approve the formation of a new county sanitation district; and

2. use the terms of this Agreement as a guide for the preparation of the conditions of formation for the new county sanitation district; and

3. require, as conditions of the formation, that the new county sanitation district make formal application to the Districts:

a. to become signatory to the JAA, and

b. to enter into an agreement substantially similar to the existing SCVJSSA regarding the ownership and operation of wastewater management facilities in the Santa Clarita Valley.

NLFC shall be responsible for preparing and submitting any necessary documents requested by the Board of Supervisors to complete the petition for formation. The Districts shall provide assistance and information to LAFCO and NLFC as requested.

B. The agreement specified in Section I.A.3.b shall be functionally the same as the existing SCVJSSA attached as Exhibit A hereto, except that it shall contain the following modifications:

1. All existing and future treatment and disposal facilities and all existing and future conveyance facilities that carry wastewater from more than one District shall be jointly owned by all of the parties signatory to the modified SCVJSSA. These jointly owned facilities shall be referred to as the Santa Clarita Valley Joint Sewerage System (SCVJSS). The rights to use these facilities shall be on a first-come, first-serve basis, except as provided below:

   a. The new county sanitation district, at its own cost and expense, shall have the right to build and use up to 6.9 mgd of capacity at the site identified in the Newhall Ranch Specific Plan for the proposed Newhall Ranch Water Reclamation Plant (NRWRP).

   b. The new county sanitation district shall have limited rights to use capacity at the Valencia Water Reclamation Plant (VWRP), pursuant to Section II.

2. The cost of operating the jointly owned facilities shall be apportioned to each party on the basis of the number of sewage units, as that term is defined in the JAA, attributable to that party's use of the wastewater management facilities. Any costs not attributable to jointly owned facilities shall be the sole responsibility of the party that caused those costs to be incurred.

3. The decision to expand the capacity of the NRWRP shall only require the approval of one party to the modified SCVJSSA until the new county

sanitation district uses 6.9 mgd of capacity in the SCVJSS. After the new county sanitation district uses 6.9 mgd of capacity in the SCVJSS, the decision to expand the capacity of the NRWRP shall require majority approval.

II. INITIAL CONNECTION OF DISCHARGERS IN THE NEW COUNTY SANITATION DISTRICT

A. NLFC shall be allowed to connect parcels located within the new county sanitation district to the SCVJSSA with a discharge of up to 3,000 capacity units, upon compliance with all of the following conditions:

1. The new county sanitation district is formed.

2. The new county sanitation district becomes a party to both the JAA and modified SCVJSSA with the consent of the other parties thereto.

3. The new county sanitation district has enacted appropriate connection fee ordinances providing for the new county sanitation district to pay into the SCVJSS Capital Improvement Fund the same amount for each discharger within the new county sanitation district that District No. 32 would have paid into the SCVJSS Capital Improvement Fund if that discharger had been located within District No. 32.

4. The new county sanitation district and NLFC have executed an agreement pertaining to the initial management expenses pursuant to Section VIII.

5. NLFC conveys title to 21.5 acres of land to the new county sanitation district for siting the NRWRP.

B. In addition to the 3,000 capacity units allowed in Section II.A, NLFC shall be allowed to connect additional parcels with a discharge of up to 3,000 capacity units provided that:

1. The conditions of Section II.A are fulfilled.

2. An NPDES permit has been obtained for the initial 2 mgd stage of the NRWRP, pursuant to the terms of Section III of this Agreement.

III. NEWHALL RANCH WATER RECLAMATION PLANT

A. NLFC will design and obtain necessary permits from the Regional Water Quality Control Board for, and fund construction of, the initial 2 mgd stage of the NRWRP, including any necessary agreements or facilities for effluent disposal.

B. The initial 2 mgd stage of the NRWRP will be designed and permitted to have an effluent disposal system independent of the existing Districts' facilities, discharging either directly to the river or other permitted place of discharge or reuse. When the initial 2 mgd stage of the NRWRP is designed and permitted, a plan will be put in place to guarantee the discharge of up to 6.9 mgd of effluent to be disposed or reused in a manner similar to the first 2 mgd. This plan will be subject to the review and approval of the Districts' Chief Engineer and General Manager (Chief Engineer). This plan will include an agreement with the reclaimed water purveying agency to take effluent from the NRWRP. The Districts will allow, under terms agreeable to the Districts, available reclaimed water from the VWRP to be taken by a reclaimed water purveying agency for use on Newhall Ranch.

C. Upon completion of the construction of the initial 2 mgd stage of the NRWRP and a successful demonstration of its operation, the NRWRP shall become a jointly owned facility of the Districts and the new county sanitation district, and the Districts and the new county sanitation district will assume ownership of and responsibility for the operation and maintenance of the NRWRP and for future expansions of the NRWRP.

D. NLFC agrees to compensate the SCVJSS for any costs incurred by the SCVJSS to make the initial 2 mgd stage of the NRWRP compliant with any of the permits required for the operation and maintenance thereof if, upon initial start up and operation, it is unable to meet its permit conditions or requirements. The SCVJSS shall be responsible for bringing the NRWRP into compliance with any future changes to permit conditions or requirements.

IV. MANAGEMENT OF THE SANTA CLARITA VALLEY JOINT SEWERAGE SYSTEM

A. Upon acceptance of the initial 2 mgd stage of the NRWRP by the SCVJSS, the Chief Engineer shall have the right to divert flow from the Districts to the NRWRP as he deems necessary for the proper and economic operation of the NRWRP and the SCVJSS as a whole.

B. Upon the initial 2 mgd stage of the NRWRP becoming a SCVJSS facility, the SCVJSS shall be responsible for future expansions of the NRWRP except that the new county sanitation district will retain its right to construct up to 6.9 mgd of capacity at the NRWRP site at its own cost and expense pursuant to Section 1.B. The SCVJSS shall be relieved of this responsibility to construct additional capacity for the use of the new county sanitation district if insufficient disposal capacity exists for the proposed NRWRP expansion.

V. CONVEYANCE FACILITIES

A. NLFC will design, fund, and construct a sewer system to collect and convey all of the wastewater to be generated within the new county sanitation district to the proposed NRWRP.

B. NLFC will design, fund, and construct all sewers, pumping plants, or force mains required to convey any flow generated within the new county sanitation district that will be treated at the VWRP.

C. The Chief Engineer will identify those sewers, pumping plants, and force mains that will be owned and operated by the new county sanitation district or the SCVJSS during the review of the design for the sewer system. Sewers that will be owned by the new county sanitation district or the SCVJSS will be designated trunk sewers. Generally, an individual district or the SCVJSS will assume ownership of and operation and maintenance responsibilities for any sewers 18 inches in diameter and larger. The design of all sewers, pumping plants, and force mains to be owned by the new county sanitation district or the SCVJSS shall be subject to the design review and approval of the Chief Engineer.

D. NLFC will be responsible for operation and maintenance of the local sewers, pumping plants, and force mains, or transfer these responsibilities to another party willing to assume that responsibility.

E. If, during the design review, the Chief Engineer determines that the trunk sewers, pumping plants, or force mains should be increased in size for uses other than those in the new county sanitation district, then NLFC shall construct these facilities with the increased capacity specified by the Chief Engineer, subject to the reimbursement provisions, below.

VI. REIMBURSEMENT

A. NLFC shall be reimbursed for its costs for the design, permitting, and construction of the initial 2 mgd stage of the NRWRP and the conveyance system.

1. NLFC's reimbursable costs for the initial 2 mgd stage of the NRWRP shall be determined as follows:

   a. The costs for the construction shall be based on the amount approved by Chief Engineer.

   b. The costs for the design and planning (including permitting) shall be based on the amount of the construction contract and the allowance factors utilized under the State Revolving Fund (SRF) loan program.

   c. The costs for the site improvements shall be based upon the difference between the cost of the site improvements for all 21.5 acres and the cost to improve the site to accommodate a 6.9 mgd treatment facility.

2. The costs for the conveyance facilities related to upsizing requested by the Chief Engineer shall be based on the difference between the actual cost of the conveyance facilities as designed and constructed, and the Chief Engineer's determination of the estimated cost for design and construction of the conveyance facilities if upsizing had not been requested by the Chief Engineer.

3. The costs for the conveyance system not related to upsizing shall be based upon the difference between the actual cost of the conveyance system designed and constructed, excluding the cost of upsizing, as calculated in accordance with Section VI.A.2, and the Chief Engineer's determination of the estimated cost to design and construct a conveyance system to serve the initial stages of the Newhall Ranch Specific Plan development. The size of the conveyance system required to support the initial stages of the Newhall Ranch Specific Plan Development shall be determined by the parties hereto, including the new county sanitation district if already formed, following the completion of design.

4. The reimbursement of costs and expenses related to the construction of the initial 2 mgd stage of the NRWRP shall be in the form of:

   a. A cash payment equivalent to the amount of the connection fees paid for up to 3,000 capacity units connected pursuant to Section II.B during construction of the initial 2 mgd stage of the NRWRP. This payment shall be made at the time the NRWRP becomes a jointly owned facility of the new county sanitation district and the Districts pursuant to Section III.C.

   b. Credits to be applied against connection fees imposed by the new county sanitation district on connections made after the NRWRP becomes a jointly owned facility of the new county sanitation district and the Districts pursuant to Section III.C.

   For purposes of this Section, no credit or reimbursement shall be given for costs that were financed pursuant to Section VII of this Agreement.

5. The reimbursement for costs related to upsizing of the conveyance system requested by the Chief Engineer shall be, at the option of NLFC, in the form of either a credit to be applied against future connection fees imposed by the new county sanitation district or a cash payment.

6. The reimbursement for costs for conveyance facilities for the new county sanitation district not related to upsizing requested by the Chief Engineer

shall be in the form of a credit to be applied against future connection fees imposed by the new county sanitation district.

VII. STATE REVOLVING FUND LOANS

NLFC and the Districts agree to cooperate and use their best efforts to obtain low interest State Revolving Fund (SRF) loans to fund construction of the initial 2 mgd stage of the NRWRP. NLFC shall guarantee repayment of debt service if the connection fee revenue in the new county sanitation district is insufficient to meet the annual debt repayment.

VIII. INITIAL MANAGEMENT COSTS FOR THE NEW COUNTY SANITATION DISTRICT

NLFC agrees to pay the new county sanitation district for all costs related to operation of the new county sanitation district until the new county sanitation district begins collecting user fees adequate to fund its ongoing obligations. Terms for this agreement, including reimbursement, will be delineated in an agreement between NLFC and the new county sanitation district following its formation.

IX. CHLORIDE CONTROL MEASURES

A. NLFC shall support the efforts of both the new county sanitation district and the Districts (i) to implement ordinances regarding the control of chlorides; (ii) to defend the ordinances regulating chlorides against any legal or political challenges; and (iii) to implement or change chloride related legislation or regulatory requirements. As part of its assistance, NLFC shall not unreasonably withhold approval for the Districts to utilize lands owned by NLFC for a chloride threshold evaluation study.

B. NLFC will use its best efforts to assist in providing information to residential and commercial properties in the new sanitation district concerning regulations and issues pertaining to the control of chlorides.

X. FAILURE TO IMPLEMENT/COMPLY

A. If, following formation of the new county sanitation district, the new county sanitation district fails to make formal application to become signatory to the JAA, or refuses to sign the JAA, this Agreement shall terminate, except that NLFC shall

continue to be responsible for any cost or expense incurred by the Districts pursuant to the terms of the Reimbursement Agreement. If any of the other County Sanitation Districts of Los Angeles County refuse to allow the new county sanitation district to become a party to the JAA, this Agreement shall terminate, except that NLFC shall continue to be responsible for any cost or expense incurred by the Districts pursuant to the terms of the Reimbursement Agreement and the Districts shall continue to be responsible for reimbursing NLFC for any cost or expense incurred on behalf of the Districts pursuant to Section V.E of this Agreement.

B. If, following the formation of the new county sanitation district, the new county sanitation district fails to request that the Districts enter into an agreement substantially similar to the SCVJSSA or refuses to sign such an agreement, this Agreement shall terminate, except that NLFC shall continue to be responsible for any cost or expense incurred by the Districts pursuant to the terms of the Reimbursement Agreement. If the Districts refuse to enter into such an agreement with the new county sanitation district, this Agreement shall terminate, except that NLFC shall continue to be responsible for any cost or expense incurred by the Districts pursuant to the terms of the Reimbursement Agreement and Districts shall continue to be responsible for reimbursing NLFC for any upsizing cost or expense incurred on behalf of the Districts pursuant to Section V.E of this Agreement.

C. If NLFC fails to obtain an NPDES permit pursuant to the terms of Sections II and III of this Agreement, then NLFC shall pay to the Districts the full amount of the fees required to annex to District No. 32 any parcels that have been connected pursuant to Section II, and this Agreement shall terminate except that the Districts shall retain title to any lands transferred pursuant to Section II.

XI. EXCLUSIVE REMEDY

The parties to this agreement understand and agree that if the Districts or the SCVJSSA are unable, for reasons beyond their control, to construct further stages of the NRWRP, NLFC's sole and exclusive remedy is to construct or request the new county sanitation district to construct any needed capacity for the Newhall Ranch Specific Plan development at the NRWRP.

XII. ASSIGNMENT

Any of the parties may assign their interests in this Agreement to a third party with the approval of the other parties. This approval shall not be unreasonably withheld. NLFC hereby consents and approves any assignment by the Districts to any of the other County Sanitation Districts of Los Angeles County.

XIII. INDEMNIFICATION

A. NLFC agrees that neither the Districts, nor any of their officers, directors, or employees shall be liable for any costs or expenses arising out of NLFC's performance of its duties under the terms of this Agreement including, but not limited to, the formation of the new county sanitation district, the transfer of property related to the siting of the NRWRP, and the design and permitting of the initial 2 mgd stage of the NRWRP. NLFC shall fully indemnify and hold the Districts harmless from any and all claims, actions, liabilities, costs or expenses arising out of NLFC's performance of this Agreement, including reasonable attorneys fees, losses and/or liabilities incurred by the Districts as a result thereof.

B. NLFC agrees that neither the Districts, nor the SCVJSSA, nor any of their officers, directors, or employees shall have any liability to NLFC or its successors or assigns or otherwise have any responsibility to NLFC if, after assuming ownership and operation of the NRWRP, the SCVJSS is unable, for reasons beyond its immediate control, to construct further expansions of the NRWRP. NLFC shall hold the Districts and the SCVJSS harmless from any and all costs, expenses, and/or losses resulting from NLFC's inability to develop its holdings within the new county sanitation district under such conditions.

C. The Districts agree that neither NLFC, nor any of its officers, directors, or employees shall be responsible for any claims, liability, demands or actions arising out of the operation of the NRWRP once the SCVJSS has assumed ownership. It is also understood and agreed that the Districts shall fully indemnify and hold NLFC harmless from any and all costs or expenses incurred, including reasonable attorneys fees, losses and/or liability attributable to any such claims resulting from the Districts' actions unless otherwise provided for in this Agreement.

XIV. MISCELLANEOUS

A. Entire Agreement. This Agreement (i) contains, and is intended as, a complete statement of all the terms of the arrangements of the parties with respect to the matters provided for herein; (ii) supersedes any previous agreements, written or oral, and understandings among the parties with regard to those matters; and (iii) cannot be changed or terminated orally. Notwithstanding the foregoing, the parties acknowledge that the provisions of any other valid mutual written agreements shall remain in full force and effect.

B. Waiver. Any party may waive compliance by the other party with respect to any provisions of this Agreement. No waiver of any provision shall be construed as a waiver of any other provision. No waiver shall be construed as an ongoing waiver with respect to subsequent events unless it expressly so provides. Any waiver must be in writing, signed by the waiving party and recite the provisions being waived.

C. Binding Effect. Subject to the terms and conditions elsewhere in this Agreement, this Agreement shall be binding upon and inure to the benefit of the parties and their respective successors and assigns. Except for rights created pursuant to the indemnity provisions of this Agreement, nothing in this Agreement shall create or be deemed to create any third party beneficiary rights of any person or entity not a party to this Agreement.

D. Duplicate Originals. This Agreement shall be executed in duplicate originals.

E. Further Assurances. Upon request by any party, the parties covenant, at any time and from time to time, to cooperate reasonably with each other and take all such further actions, including without limitation the execution and filing of additional instruments or documents, as may be reasonably necessary to carry out the intent, purposes and terms of this Agreement. Such cooperation shall be exercised in good faith with respect to any additional instruments or documents that may be necessary to carry out the intent, purposes, and terms of this Agreement, and any such additional instruments or documents shall be reviewed on their own merits, and not as a basis for the introduction of unrelated matters, nor require any additional consideration.

F. Applicable Law. This Agreement shall be construed in accordance with the laws of the State of California. Venue for any proceeding shall be the Superior Court of Los Angeles County.

G. Incorporation of Recitals. The Recitals to this Agreement are incorporated herein by this reference and are acknowledged by the parties to be true and correct.

H. Action By The Chief Engineer. Except as otherwise provided in this Agreement, the Chief Engineer shall have the authority to take all actions on behalf of the Districts in connection with any approvals or actions required of or by the Districts under this Agreement.

I. No Joint Venture. This Agreement does not constitute and shall not be construed as constituting a partnership or joint venture between the Districts and NLFC in any matter whatsoever.

J. Construction/Interpretation. The parties to this Agreement acknowledge that they have been represented by counsel in respect of the negotiation and drafting of this Agreement, and that no term, covenant or condition of this Agreement shall be construed or interpreted by reference to the extent to which either party participated in the drafting of the Agreement, or any party thereof.

K. Attorneys' Fees. In any action or proceeding arising under this Agreement, or to enforce the provisions of this Agreement, the parties shall bear their own attorneys' fees and costs.

L. Amendments in Writing/Cooperation. This Agreement may be amended or modified, but only in writing, duly executed by the parties to this Agreement. The parties shall cooperate in good faith with respect to any amendment or modification proposed in order to clarify the intent and application of this Agreement, and shall treat any such proposal on its own merits, and not as a basis for the introduction of unrelated matters.

M. Time of Essence. Time is of the essence in the performance of the provisions of this Agreement.

N. Notices. All notices, requests, demands, or other communications under this Agreement shall be in writing (except for any emergency notice, which must be followed by written notice as soon as reasonably possible but in no event more than twenty-four (24) hours from the oral notice) and sent to the addressees of each party. Notice will be sufficiently given for all purposes as follows:

PERSONAL DELIVERY: When personally delivered to the recipient. Notice is effective on delivery.

FIRST-CLASS MAIL: When mailed first-class to the last address of the recipient known to the party giving notice. Notice is effective five mail delivery days after it is deposited in a United States Postal Service office or mailbox.

CERTIFIED MAIL: When mailed certified mail, return receipt requested. Notice is effective on receipt, if a return receipt confirms delivery.

OVERNIGHT DELIVERY: When delivery by an overnight delivery service such as Federal Express, charged prepaid or charged to the sender's account. Notice is effective on delivery, if delivery is confirmed by the delivery service.

FACSIMILE DELIVERY: When transmitted by facsimile to a party's fax number, a written notice shall be contemporaneously sent by personal delivery, first-class mail, certified mail or overnight delivery. A facsimile notice is effective if received within the hours of 8:00 a.m. to 4:00 p.m. (local time of the recipient) during the business day and a transmission report is generated reflecting the accurate transmission of this facsimile. If a facsimile is received at any other time, it shall be deemed received the next business day. A "business day" shall be Monday through Friday except for any holiday recognized by the Districts.

Addresses for purpose of giving notice are as follows:

DISTRICTS:

Mr. James F. Stahl
Chief Engineer and General Manager
County Sanitation Districts of Los Angeles County
1955 Workman Mill Road
Whittier, CA 90601-1400
Post Office Box 4998
Whittier, CA 90607-4998
Facsimile No.: (562) 695-8660

COPY TO:

B. Richard Marsh, Esq.
Daniel V. Hyde, Esq.
Lewis, D'Amato, Brisbois & Bisgaard LLP
221 North Figueroa Street, Suite 1200
Los Angeles, CA 90012
Facsimile No.: (213) 250-7900

NLFC:

Steven Zimmer
Vice President
The Newhall Land and Farming Company
23823 Valencia Boulevard
Valencia, CA 91355
Facsimile No.: (661) 288-1052

IN WITNESS HEREOF, THE NEWHALL LAND AND FARMING COMPANY AND COUNTY SANITATION DISTRICTS NOS. 26 AND 32 OF LOS ANGELES COUNTY hereby cause this Agreement to be executed and agree to be bound by the terms and conditions set forth herein.

THE NEWHALL LAND AND FARMING COMPANY (A CALIFORNIA LIMITED PARTNERSHIP)

By: Newhall Management Limited Partnership, a California limited partnership, Its Managing General Partner

By: Newhall Management Corporation, a California corporation, Its Managing General Partner

By: _____
Its Vice President

By: _____
Its Assistant Secretary

ATTEST:

COUNTY SANITATION DISTRICT NO. 26 OF LOS ANGELES COUNTY

By: M. Alma Horvath
Secretary

By: Jo Anne Darcy
Chairperson, Board of Directors
JAN 0 9 2002

APPROVED AS TO FORM:
LEWIS, D'AMATO, BRISBOIS & BISGAARD, LLP

By: Daniel V. Hyde
District Counsel

ATTEST:

COUNTY SANITATION DISTRICT NO. 32 OF LOS ANGELES COUNTY

By: M Alma Horvath
Secretary

By: Jo Anne Darcy
Chairperson, Board of Directors
JAN 0 9 2002

APPROVED AS TO FORM:
LEWIS, D'AMATO, BRISBOIS & BISGAARD, LLP

By: Daniel V. Hyde
District Counsel

APPENDIX A

Joint Powers Agreement for a Regional Sewerage System
in the Santa Clarita Valley

CCRSTR # 2694

JOINT POWERS AGREEMENT
FOR A REGIONAL SEWERAGE SYSTEM
IN THE SANTA CLARITA VALLEY

THIS AGREEMENT is made and entered into this 8th day of May, 1984 by and between

COUNTY SANITATION DISTRICT NO. 26 OF LOS ANGELES COUNTY, hereinafter referred to as "DISTRICT NO. 26", and

COUNTY SANITATION DISTRICT NO. 32 OF LOS ANGELES COUNTY, hereinafter referred to as "DISTRICT NO. 32".

WITNESSETH

WHEREAS, pursuant to the provisions of the County Sanitation District Act of 1923 (Chapter 3, Part 3, Division 5, of the Health and Safety Code of the State of California), DISTRICT NO. 26 and DISTRICT NO. 32, hereinafter referred to collectively as the "Districts", have heretofore acquired necessary lands and rights-of-way and have carried out the construction of a sewage collection and treatment system to serve certain areas in the Santa Clarita Valley; and

WHEREAS, the Districts and their potential areas for annexation and expansion lie in the same drainage basin; and

WHEREAS, the projected wastewater flow from future development in the drainage basin exceeds the existing capacity of the treatment plants in both Districts; and

WHEREAS, the nature of the land adjacent to the Saugus Water Reclamation Plant (District No. 26) limits the area available for construction of expanded treatment works; and

WHEREAS, the Valencia Water Reclamation Plant (District No. 32) site has suitable land adequate in size for the construction of a regional treatment plant that would meet the needs of both Districts; and

WHEREAS, the wastewater flows attributable to both Districts can be managed most efficiently and economically through the creation and operation of a regional sewerage system; and

WHEREAS, the Districts entered into an Interim Joint Sewage Disposal Agreement dated April 24, 1979, which provided for the short-term disposal needs of both Districts and provided for the preparation of a facilities plan to assist in the development of a long-term regional wastewater treatment and disposal plan; and

WHEREAS, the Districts prepared a facilities plan dated April 1980, setting forth the long-term needs and solutions for the provision of adequate wastewater conveyance, treatment, and disposal facilities for both Districts; and

WHEREAS, it is the intent of the Districts to jointly finance, construct, and operate the regional conveyance, treatment, and disposal facilities necessary to provide for the long-term needs of the Districts;

NOW, THEREFORE, it is hereby agreed as follows:

1. DEFINITIONS

As used in this Agreement, the following terms shall have the meanings herein set forth:

a. Capital Improvement Fund

A separate fund for which any monies deposited therein and any interest derived thereon are to be used exclusively for payment of capital facilities, either local sewerage facilities or Santa Clarita Valley Joint Sewerage System facilities, as hereinafter defined.

b. Capital Costs

All expenditures made for the acquisition of or addition to fixed assets and all allocations to a fund for such purposes.

c. Chief Engineer

The Chief Engineer and General Manager of District No. 26 and District 32 or his duly authorized deputy or agent.

d. Connection Fee Ordinance(s)

Connection fee ordinance(s) shall refer to the "Connection Fee Ordinance for County Sanitation District No. 26 of Los Angeles County" adopted January 12, 1984 and as thereafter amended; and to the "Connection Fee Ordinance for County Sanitation District No. 32 of Los Angeles County", adopted January 12, 1984 and as thereafter amended.

e. Local Costs

Costs expended for the acquisition, construction, repair, replacement, reconstruction, and operation and maintenance of local sewerage facilities, as hereinafter defined.

f. Local Sewerage Facilities

Those sewer lines or portions thereof, pumping plants, and other sewerage facilities utilized solely by one District, and which serve to benefit only that District, and which have not been defined as part of the Santa Clarita Valley Joint Sewerage System, as hereinafter defined.

g. Joint Costs

Costs expended for the acquisition, construction, repair, replacement, reconstruction, and operation and maintenance of the Santa Clarita Valley Joint Sewerage System, as hereinafter defined.

h. Operation and Maintenance

The operation and maintenance of, and the repair, replacement, and upkeep of all sewerage facilities inclusive of all expenses incidental thereto.

i. Santa Clarita Valley Joint Sewerage System

The system of wastewater collection, treatment, and disposal facilities that provides a benefit to more than one District, which will initially consist of the facilities defined in Exhibit I; and as those facilities may from time to time be modified or expanded.

j. Service Charge Ordinance(s)

Service Charge Ordinance(s) shall refer to the "1983 Service Charge Ordinance of County Sanitation District No. 26", adopted April 12, 1983, and as thereafter amended; and to the "1983 Service Charge Ordinance of County Sanitation District No. 32", adopted April 12, 1983, and as thereafter amended.

k. Sewage Unit

The average daily quantity of sewage flow and strength from a single family home measured in terms of flow, chemical oxygen demand, and suspended solids as defined in the Service Charge Ordinance.

l. Surcharge Revenue

The revenue derived from the Wastewater Treatment Surcharge Program as provided in the Wastewater Ordinance, as hereinafter defined.

m. Wastewater Ordinance

An ordinance providing for the administration of an industrial wastewater control system, for regulating sewer construction or use, for the imposition of a permit requirement for industrial wastewater dischargers, for the prohibition or pretreatment of industrial wastewaters, for the imposition of fees and charges for the distribution of revenue and for other methods of controlling and regulating the discharge of wastewaters, adopted April 1, 1972, as amended July 1, 1980 and as thereafter amended.

2. ESTABLISHMENT OF AND JOINT PARTICIPATION IN A REGIONAL SEWERAGE SYSTEM

The parties hereto shall participate jointly according to proportions hereinafter specified in the acquisition, construction, ownership, mainte-

nance, and operation of the system of main trunk sewers, pumping plants, treatment plants, and sanitation works herein referred to as the "Santa Clarita Valley Joint Sewerage System" including the parts heretofore acquired or which may hereafter be acquired and/or constructed; and to pay for such ownership, maintenance, and operation in the manner hereinafter specified.

District No. 26 and District No. 32, together with other County Sanitation Districts of Los Angeles County, entered into an Amended Joint Administration Agreement on July 1, 1980, providing for a single and centralized administrative organization. District No. 26 and District No. 32 agree that this Joint Powers Agreement shall be administered and executed pursuant to the provisions of said Amended Joint Administration Agreement.

All contracts for construction of Santa Clarita Valley Joint Sewerage System facilities shall be awarded and administered by the District in which the facilities will be located. However, no such contract shall be awarded without the approval of both Districts.

3. CAPITAL IMPROVEMENT FUND

Each District shall establish a capital improvement fund, hereinafter referred to as the "Local Capital Improvement Fund", which shall be solely owned and maintained by that District. The Districts shall also establish a capital improvement fund, hereinafter referred to as the "Joint Capital Improvement Fund," which shall be jointly owned and maintained by both Districts.

All connection fees, as provided for in the connection fee ordinances, shall be apportioned into local and joint shares. The allocation percentages for the local and joint shares shall be determined annually by the Chief Engineer and shall be based upon the percentages of the connection fee which are attributable to local and joint costs, respectively. The local share shall be deposited into the Local Capital Improvement Fund of the District which collected the funds and the joint share shall be deposited into the Joint Capital Improvement Fund. All interest shall remain with the capital improvement fund from which it was derived.

Any monies on deposit in the existing local capital improvement funds at the execution of this Agreement shall likewise be apportioned between the local and joint capital improvement funds established by this Agreement. The allocation percentages for this procedure shall be the same as those in effect at the time of execution of this Agreement.

No monies shall be withdrawn from the Joint Capital Improvement Fund without the approval of both Districts.

4. ALLOCATION OF COSTS ATTRIBUTABLE TO THE SANTA CLARITA VALLEY JOINT SEWERAGE SYSTEM

a. Operation and Maintenance

All costs for operations and maintenance which are attributable to the Santa Clarita Valley Joint Sewerage System shall be apportioned

to each District in the ratio that the number of sewage units originating from each District bears to the aggregate number of sewage units in both Districts. The Chief Engineer and General Manager shall determine annually the total number of sewage units attributable to each District.

b. Original Capital Construction

Each District agrees that certain facilities, described in Exhibit 2 hereof, would have been constructed irrespective of this Agreement. Both Districts agree to construct those same facilities and allocate costs for such facilities not funded through the Joint Capital Improvement Fund as set forth in Exhibit 2.

c. Expansion Capital

All capital costs for projects which increase the capacity of the Santa Clarita Valley Joint Sewerage System but do not provide a greater level of treatment or disposal shall be financed through the use of the Joint Capital Improvement Fund to the extent possible. If the cost of such a project exceeds the resources available in the Joint Capital Improvement Fund, the remaining cost shall be apportioned to each District in the ratio that the number of sewage units attributable to each District bears to the aggregate number of sewage units in both Districts.

d. Upgrade Capital

All capital costs for projects which provide a greater level of treatment or disposal but do not increase the capacity of the Santa Clarita Valley Joint Sewerage System shall be apportioned to each District in the ratio that the number of sewage units attributable to each District bears to the aggregate number of sewage units in both Districts. If the capacity of the Santa Clarita Valley Joint Sewerage System exceeds the requirements of both Districts at the time a contract for an upgrade capital project is awarded, the costs associated with the excess capacity shall be financed through the use of the Joint Capital Improvement Fund to the extent possible.

5. LOCAL SEWERAGE FACILITIES

All costs associated with local sewerage facilities shall be the responsibility of the District for whose benefit such costs are incurred, and shall not be allocated in any manner to the other District.

6. ALLOCATION OF REVENUES

a. Grants

All monies received as grants to be expended for capital projects associated with the Santa Clarita Valley Joint Sewerage System shall be apportioned to each District in the same ratio that the cost of

said projects is apportioned to each District as provided for in Article 4 hereof.

b. Joint Contract Revenue

All revenue which is derived from a Joint Contract for the disposal of sewage, brine, or industrial wastewater originating from outside of the Districts' boundaries, as hereinafter defined, shall be apportioned into joint and local shares based upon the ratio of the local and joint costs to the total costs incurred due to said contract. The allocation percentages shall be determined annually by the Chief Engineer. The local share of said revenue shall be given to the District which has incurred local costs in conjunction with the execution of said contract. The joint share shall be apportioned to each District according to the ratio of its obligation to pay for the operation and maintenance costs of the system as provided in Article 4 hereof.

c. Other Joint Revenues

All revenue received from the sale of sludge, effluent, surplus energy or other by-products or which otherwise arises out of the ownership or operation of any Santa Clarita Valley Joint Sewerage System facility shall be apportioned to each District according to the ratio of its obligation to pay for operation and maintenance cost of the system as provided for in Article 4 hereof.

d. Local Revenue

All other revenue including, but not limited to, ad valorem taxes, surcharge revenue derived pursuant to the Wastewater Ordinance, service charges, annexation fees, investment income, payments from local contracts as hereinafter defined, and local capital improvement fund monies shall remain with the District from which it originates.

7. DISPOSAL OF SEWAGE ORIGINATING OUTSIDE DISTRICTS

Each District undertakes and agrees that it will not permit or knowingly countenance the use of the Santa Clarita Valley Joint Sewerage System, directly or indirectly, for the disposal of sewage, brine, or industrial wastes originating outside of its territorial limits except in the manner, to the extent, and on the conditions agreed upon by both Districts. In the event, however, that the anticipated sewage units from said outside source do not exceed one-half of one percent (.5%) of the aggregate number of sewage units originating from both Districts, such agreement by both Districts shall not be required. Any disposal contract for which agreement is not required shall be considered a local contract and the sewage units received from such a contract shall be regarded as having originated from the District with which the contract is made for the purpose of determining the total number of sewage units attributable to that District.

Any disposal contract heretofore or hereafter entered into, for which the actual or anticipated sewage units exceeds one-half of one percent (0.5%)

of the aggregate number of sewage units originating from both Districts shall be considered a joint contract. The sewage units attributable to a joint contract shall be apportioned to each District in the ratio that the number of sewage units originating in each District bears to the aggregate number of sewage units originating in both Districts.

8. TITLE TO REAL PROPERTY

Fee title to real property required for the Santa Clarita Valley Joint Sewerage System for the use and benefit of both Districts shall be acquired by, and in the name of, the District in which the said real property is located, whether such acquisition is by purchase, condemnation, gift, or otherwise.

9. SETTLEMENT OF DISPUTE OR CONTROVERSY

Should any dispute or controversy arise in connection with the books, records, or accounts of either District pertaining to the Santa Clarita Valley Joint Sewerage System or in connection with the acquisition, construction, maintenance, operation, repair, reconstruction, or enlargement of the Santa Clarita Valley Joint Sewerage System, or in connection with any of the affairs or operation thereof, the Boards of Directors of the Districts shall appoint a mutually agreed upon arbitrator, and the determination of said arbitrator shall be binding and conclusive.

10. WITHDRAWAL

This Agreement and the Santa Clarita Valley Joint Sewerage System provided hereby shall continue in effect so long as it shall be in the interest of both Districts. If at any time it should cease to be in the interest of either District to continue as a party to this Agreement, and its Board of Directors shall so find and declare by resolution, it may withdraw herefrom and terminate its obligation hereunder after sixty (60) days notice in writing of its intention to do so, and by the full and complete discharge of all its debts and obligations hereunder for which it shall be or become liable up to the date of its withdrawal.

Disposition of all assets of the Santa Clarita Valley Joint Sewerage System shall be in proportion to the number of sewage units attributable to each District on the date of withdrawal.

11. EFFECTIVE DATE

This Agreement shall become effective on July 1, 1984.

12. TERMINATION OF PRIOR AGREEMENT

It is mutually agreed that, upon the effective date of this Agreement; the Interim Joint Sewage Disposal Agreement, dated April 24, 1979, shall terminate and be of no further force and effect but the parties to such Agreement shall nevertheless pay and discharge all obligations previously accrued thereunder.

IN WITNESS WHEREOF, the parties have executed this Agreement the date and year above set forth.

ATTEST:

By _____
Secretary

COUNTY SANITATION DISTRICT NO. 26
OF LOS ANGELES COUNTY

By _____
Chairman

ATTEST:

By _____
Secretary

COUNTY SANITATION DISTRICT NO. 32
OF LOS ANGELES COUNTY

By _____
Chairman

APPROVED AS TO FORM:

KNAPP, GROSSMAN & MARSH

By _____
B. Richard Marsh, Counsel
for County Sanitation Districts
of Los Angeles County

EXHIBIT 1

The following facilities shall be defined as being part of the Santa Clarita Valley Joint Sewerage System:

(1) Saugus Water Reclamation Plant

(2) Valencia Water Reclamation Plant

(3) District 26 Interceptor

(4) District 32 Main, Sections 1 and 2

(5) Saugus Water Reclamation Plant Waste Activated Sludge Force Main

EXHIBIT 2

Per Article 4b of this Agreement, the facilities to be constructed are as follows:

Saugus Water Reclamation Plant (WRP) Filtration and Chlorination Facilities (Existing plant to be upgraded to advanced treatment):

- Primary sedimentation tank
- Return sludge pumps
- Waste activated sludge pump
- Filters with filter feed and backwash pump stations
- Filter waste backwash recovery tanks
- Chlorine contact tank
- Chlorination/dechlorination systems
- Alum/Polymer systems and appurtenances
- Standby power
- High pressure effluent system
- Office/lab addition

Saugus WRP Waste Activated Sludge Force Main (will convey waste activated sludge to the Valencia WRP):

- An approximately 14,900 L.F. force main

Valencia WRP Stage III Expansion (3.0 mgd expansion of plant capacity to receive additional flow):

- Influent pumps and metering
- Primary sedimentation tanks
- Aeration tank
- Final clarifier
- Return activated sludge pumps
- Waste activated sludge pumps
- Filters with filter feed pump station
- Waste backwash decant tank
- Chlorination/dechlorination equipment
- Maintenance building
- Effluent discharge line with meter
- High pressure effluent and non-potable water systems

Saugus/Valencia Solids Processing Facilities (construction of sludge digestion and dewatering facilities to handle solids from both District 26 and District 32):

- Dissolved air flotation thickener
- Anaerobic digesters
- Digester cleaning decant facility with submersible pump
- Digested sludge holding tanks
- Frame plate filter press and appurtenances

EXHIBIT 2 (cont.)

Filtrate equalization tank
Energy recovery system
Truck Maintenance Facility
Sludge Hauling Trucks

The Districts further agree to allocate the costs of these facilities according to the following percentages:

FACILITY	DISTRICT NO. 26 PERCENTAGE	DISTRICT NO. 32 PERCENTAGE
Saugus WRP Filtration and Chlorination Facilities	100%	0%
Saugus WRP Activated Sludge Force Main	100%	0%
Valencia WRP Stage III Expansion (2.0 mgd which is grant fundable)	100%	0%
Valencia WRP Stage III Expansion (1.0 mgd which is not grant fundable)	55%	45%
Anaerobic Digesters at Valencia WRP (7.0 mgd which is grant fundable)	100%	0%
Anaerobic Digesters at Valencia WRP (1.0 mgd which is not grant fundable)	55%	45%
Solids Processing Facility (excluding anaerobic digesters)	60%	40%
Draft Tubes for Existing Valencia WRP Digesters	0%	100%

Geosyntec Memorandum – LID Water Quality Analysis Results for Landmark Village dated May 17, 2011

Geosyntec consultants

1111 Broadway, 6th Floor
Oakland, California 94607
PH 510.836.3034
FAX 510.836.3036
www.geosyntec.com

Memorandum

Date: 17 May 2011

To: Matt Carpenter and Alex Herrell, Newhall Land

From: Lisa Austin, Aaron Poresky, and Kelly Havens, Geosyntec Consultants

Subject: LID Water Quality Analysis Results for Landmark Village

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BACKGROUND

As described in the Landmark Village Draft Environmental Impact Report (EIR) Section 4.3 Water Quality, and Appendix 4.3, Landmark Village Water Quality Technical Report, the *Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan* (Geosyntec, 2008) sets forth the urban runoff management program that will be implemented for the Project. As indicated in the *Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan*, Project Design Features (PDFs) incorporated into the Landmark Village Project (the Project) to address water quality and hydrologic impacts include site design, Low Impact Development (LID), source control, treatment control, and hydromodification control best management practices (BMPs). Most of these BMPs will promote infiltration and recharge groundwater.

Site design that will promote infiltration and groundwater recharge includes clustering development within the Newhall Ranch Specific Plan area into villages. Approximately 74 percent (10,145 acres) of the Specific Plan area will remain undeveloped Open Areas.

As indicated in *Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan* and Landmark Village Draft EIR Section 4.3, Water Quality, LID BMPs that promote retention of urban runoff are included as PDFs, although the water quality modeling conducted for the impact analysis in the Draft EIR does not account for the stormwater runoff that would be retained in these BMPs. LID BMPs that meet the Project's LID BMP Performance Standard, discussed below, have been analyzed in order to quantify Project impacts with LID BMPs.

LID BMP PERFORMANCE STANDARD

A LID BMP Performance Standard conceptually similar to the LID requirements in the Ventura County MS4 Permit has been developed for the Project. The LID BMP Performance Standard is illustrated in Figure 1 and described below:

> LID project design features (PDFs) shall be selected and sized to: (1) fully retain the volume of stormwater runoff produced from a 0.75 inch storm event; and (2) reduce the percentage of Effective Impervious Area (EIA) to five percent or less of the total project area within the vesting tentative tract map and associated off-site project area. Runoff from all EIA shall be subject to treatment control measures that are selected to address the pollutants of concern and are sized to capture and treat 80 percent of the average annual runoff volume.

This LID Performance Standard would be implemented on the Project as follows:

1. Institutional, commercial, multi-family residential, recreation, and park land use parcels would implement retention or biofiltration BMPs on-site to the extent feasible. Based on an assessment of feasibility, one of three BMP strategies would be applied as outlined below:
    a. *Infiltration feasible*: If it is feasible to infiltrate all of the developed area runoff produced from the 0.75 inch design storm (i.e., soil infiltration rates are at least 0.5 inches per hour, fill depth is less than 10 feet, and no infiltration geotechnical hazards exist (such as landslides and terrace escarpments)), infiltration BMPs would be used. Infiltration BMPs include bioretention (without an underdrain), permeable pavement, infiltration galleries, infiltration basins or trenches, or an equivalent infiltration BMP.
    b. *Bioinfiltration allowable when low infiltration rates or deep fill depths are present*: If the parcel has low soil infiltration rates (i.e., the soil infiltration rate is less than 0.5 inches per hour) or the depth of fill is greater than 10 feet, but no other technical infeasibility concerns exist, bioinfiltration BMPs would be used. Bioinfiltration facilities are similar to bioretention facilities with an underdrain, but they include storage below the underdrain to maximize the volume infiltrated. These facilities would retain a portion of the runoff from the LID design storm, then biofilter the remaining runoff from the design storm.
    c. *Infiltration is not allowable*: If infiltration is technically infeasible due to geotechnical hazards or a high ground water table, then biofiltration BMPs would be used. These BMPs would biofilter the runoff produced from the LID design storm from the developed area. No areas have been identified in the Landmark Village Project area where infiltration is not allowed in any quantity.
2. Runoff from roofs, patios, and walkways in single family residential parcels would be disconnected over landscaped areas designed to fully retain the volume of runoff from the LID design storm (0.75 inch storm event). Runoff from the remaining parcel area and that

which does not infiltrate in the landscaped area would flow through the storm drain system to the regional infiltration/biofiltration facilities.

3. Runoff from roadways would be retained or biofiltered in retention or biofiltration BMPs sized to capture the LID design storm volume or flow, per the guidance in US EPA's Managing Wet Weather with Green Infrastructure: Green Streets.
4. No more than 5% of the total Project area would be treated using conventional treatment methods that address the pollutants of concern. In this case, media filters (or equivalent BMPs that address the pollutants of concern) would be sized to capture and treat 80% of the average annual runoff volume from the allowable EIA.
5. Regional or sub-regional infiltration/biofiltration facilities would also be implemented. The regional or sub-regional facilities would be designed to incorporate a biofilter in the bottom of the facility, which would allow for infiltration if feasible, with detention storage above the biofilter. The regional facilities would infiltrate or biofilter the LID design storm volume that has not been retained or biofiltered on the parcels in the area tributary to the regional facility and would provide extended detention treatment for the additional runoff volume required to provide 80% capture and treatment of the average annual runoff volume per the *Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation* Plan treatment performance standard.

METHODOLOGY

A load-based water quality model was used to estimate pollutant loads and concentrations in Project area stormwater runoff for certain pollutants of concern for pre-development conditions and post-development conditions with the LID BMPs described above. This model was coupled with hydrologic and hydraulic modules of USEPA SWMM v4.4h to quantify the volume reduction and capture efficiency of the BMPs.

Table 1 below provides a list of model inputs and the sources for these inputs. For further detail, see Appendix B of the *Landmark Village Water Quality Technical Report* (Draft EIR Appendix 4.3) and Attachment 1 to this memorandum.

Table 1: Model Input Requirements and Assumptions

Model Input	Assumption/Source
Hourly long-term rainfall record	National Climatic Data Center (NCDC) Newhall (046162) and San Fernando (047762) rain gauge data from 1969-2008
Green-Ampt soil parameters	Natural Resource Conservation Service Soil Data Mart Table 5.5.5 – Handbook of Hydrology (Maidment, ed. 2003)

<table>
<tr><th>Model Input</th><th>Assumption/Source</th></tr>
<tr><td>Land use-based imperviousness</td><td>LA County Hydrology Manual (LACDPW, 2006)</td></tr>
<tr><td>Land use-based stormwater runoff event mean concentrations</td><td>Los Angeles County 1994-2000 Integrated Receiving Water Impacts Report, 2000
Los Angeles County 2000-2001 Stormwater Monitoring Report, 2001
Ventura County Watershed Protection District
As analyzed for the Los Angeles Structural BMP Prioritization and Assessment Tool (LACDPW, City of Los Angeles, and Heal the Bay, 2008)</td></tr>
<tr><td>Volume and flow-based BMP design criteria</td><td>80% Capture of Average Annual Runoff Volume
(NRSP Sub-Regional SWMP (Geosyntec, 2008))</td></tr>
<tr><td>BMP selection criteria</td><td>Select and locate BMPs with a preference for infiltration.
Select BMPs to infiltrate the runoff volume from the 0.75-inch design storm to the extent feasible and biofilter the remaining fraction of the 80 percent capture volume.
Evaluate degree of feasibility of infiltration based on land use type, native soil infiltration rate, proposed cut and fill, depth to groundwater, presence of landslides that will remain after remedial grading, and other geotechnically- or ecologically-based constraints.</td></tr>
<tr><td>Volume reduction and LID BMPs analyzed quantitatively</td><td>Clustering (preservation of open space)
Hydrologic source controls
Distributed retention, bioinfiltration, and biofiltration BMPs
Regional infiltration, bioinfiltration, and biofiltration facilities
Media filters</td></tr>
<tr><td>Volume reduction modeling parameters</td><td>Hydrologic source controls: equal ratio of disconnected of rooftops and patios to landscaped areas receiving disconnection
Onsite BMPs:
<table>
<tr><th>Feasibility Constraint Category</th><th>Design infiltration rate (in/hr)</th></tr>
<tr><td>Category 1: Retention</td><td>0.38</td></tr>
<tr><td>Category 2: Bioinfiltration</td><td>0.25</td></tr>
</table>
Sub-regional Bioinfiltration/Biofiltration Facilities:
<table>
<tr><th>Facility Type</th><th>Design infiltration rate (in/hr)</th></tr>
<tr><td>Sub-regional Bioinfiltration Facilities (multiple)</td><td>0.25</td></tr>
<tr><td>Sub-regional Biofiltration Facility with biofiltration, extended detention and incidental infiltration</td><td>0.25</td></tr>
</table></td></tr>
</table>

Model Input	Assumption/Source
LID BMP effluent quality	ASCE/USEPA (American Society of Civil Engineers Urban Water Resources Research Council and United States Environmental Protection Agency) 2011, International Stormwater Best Management Practices Database (www.bmpdatabase.org); (Reanalysis of expanded database conducted January 2011)

The land use areas analyzed are listed in Table 2 below and illustrated in Figure 2. These land use areas are for the revised project description included in the Final EIR.

Table 2: Summary of Revised Land Use Program Analyzed

Land Use Designation	Landmark Village Project (Acres)
Commercial	27.3
Multi-Family	82.9
Open Space	32.9
Park	10.1
Recreation	5.8
Road	41.6
School	9.7
Single-Family[1]	53.9
Water Quality Facility	10.1
Total	*274.4*
Off-site Commercial (Water Tanks)	8.0
Off-site Road	98.0
Total Area	380.4

RESULTS

LID Feasibility Screening for the Project Area

An assessment of infiltration feasibility was conducted to estimate, for the Project area, which one of three BMP strategies could be applied onsite and whether the sub-regional bioinfiltration/biofiltration facilities would allow for infiltration. The Project area was analyzed by the Project Geotechnical Consultant (Alan E. Seward, 2010) using geologic information, soils information, proposed remedial grading plans, final grades, and applicable feasibility criteria from the Los Angeles County LID Standards Manual. This analysis categorized project areas into three levels of infiltration feasibility:

1. Infiltration was considered to be feasible directly from the bottom of BMPs in locations where underlying soils infiltration rates were estimated to be greater than 0.5 inches per hour and the proposed depth of compacted fill was estimated to be less than 10 feet.
2. Infiltration was considered to be feasible through the use of dry wells in locations where underlying soils infiltration rates were estimated to be greater than 0.5 inches per hour and greater than 10 feet of separation was estimated to exist from the bottom of proposed fill to the seasonally high groundwater table.
3. Infiltration was considered to be partially feasible in the remaining areas. No hazards were identified that would preclude the use of some level of infiltration.

The results of this feasibility screening are illustrated in Figure 3. Figure 4 illustrates the LID BMPs for the Project area based on the feasibility screening.

Project Impact Assessment for Modeled Pollutants of Concern

Table 3 below shows the predicted changes in stormwater runoff volume and mean annual loads for the modeled pollutants of concern for the Project area. Table 4 below shows the predicted changes in concentration in stormwater runoff for the Project area.

Table 3: Predicted Average Annual Runoff Volume and Pollutant Loads

Parameter	Units	Existing Conditions	Developed Conditions with no BMPs	Developed Conditions w/ LID	Change w/LID
Volume	acre-ft	130	384	261	131
TSS	tons/yr	37	38	12	-25
Total Phosphorus	lbs/yr	548	490	193	-355
Nitrate-N + Nitrite-N	lbs/yr	1,219	1,005	432	-787
Ammonia-N	lbs/yr	215	525	147	-68
Total Nitrogen	lbs/yr	2,137	3,118	1,277	-860
Chloride	tons/yr	3.7	8.2	5.2	1.5
Dissolved Copper	lbs/yr	10	20	8	-2
Total Lead	lbs/yr	4.5	8.4	3.0	-1.5
Dissolved Zinc	lbs/yr	63	152	45	-18
Total Aluminum[1]	lbs/yr	487	711	231	-256

[1] *BMP effectiveness studies in the International Stormwater BMP database infrequently monitor aluminum; therefore, insufficient effluent data were available to model the removal effectiveness of treatment control BMPs for this water quality constituent. In order to estimate the reduction in total aluminum load and concentration, TSS removal was used as a surrogate.*

Table 4: Predicted Average Annual Pollutant Concentrations

Parameter	Units	Existing Conditions	Developed Conditions with no BMPs	Developed Conditions w/ LID	Change w/LID
TSS	mg/L	192	72	33	-159
Total Phosphorus	mg/L	1.4	0.5	0.3	-1.1
Nitrate-N + Nitrite-N	mg/L	3.0	1.0	0.6	-2.4
Ammonia-N	mg/L	0.6	0.5	0.2	-0.4
Total Nitrogen	mg/L	6	3	2	-4
Chloride	mg/L	20	16	14	-6
Dissolved Copper	µg/L	28	20	10	-18
Total Lead	µg/L	12	8	4	-8
Dissolved Zinc	µg/L	185	146	60	-125
Total Aluminum[1]	µg/L	1282	678	323	-959

[1]*BMP effectiveness studies in the International Stormwater BMP database infrequently monitor aluminum; therefore, insufficient effluent data were available to model the removal effectiveness of treatment control BMPs for this water quality constituent. In order to estimate the reduction in total aluminum load and concentration, TSS removal was used as a surrogate.*

While runoff volume is predicted to increase, the loads of all modeled constituents with the exception of chloride are predicted to decrease and the concentrations of all modeled constituents, including chloride, are predicted to decrease under proposed Project developed conditions with LID when compared to existing conditions. The increase in runoff volume can be explained by the increase in impervious cover associated with development of the site, as well as by the decrease in infiltration capacity of existing site soils associated with the compaction of site soils during construction. The change in concentrations can be attributed to the difference in concentrations observed in monitoring data from agricultural, open space, and un-treated transportation land uses (the existing condition for the site) compared with urban land uses (representative of post-development conditions) in combination with the reductions in concentration achieved in the LID and sub-regional bioinfiltration/biofiltration BMPs. Change in pollutant load is a function of the increase in runoff volume and the relative change in pollutant concentration; if the predicted reduction in pollutant concentration is small, then the predicted runoff load may increase.

The predicted average annual TSS, nutrients, and chloride concentrations in stormwater runoff from the total modeled Project area are compared to water quality criteria in Table 5 below. Concentrations of all modeled constituents are predicted to decrease and to be below the Basin Plan water quality objectives (WQOs) and total maximum daily load waste load allocation (TMDL WLAs) benchmark criteria because of the change in land uses and the implementation of LID, biofiltration, and treatment control BMPs. In addition, all predicted concentrations are within the observed ranges of concentrations within Santa Clara River Reach 5. Based on the

comprehensive LID implementation strategy, the predicted decrease in runoff concentrations, and the comparison with benchmark criteria and instream concentrations, water quality impacts related to TSS, nutrients, and chloride would be less-than-significant with the implementation of the proposed LID BMPs.

Table 5: Comparison of Predicted Nitrogen Compound Concentrations for the Landmark Village Project Area with Water Quality Objectives, TMDLs, and Observed Concentrations in Santa Clara River Reach 5

Nutrient	Predicted Average Annual Concentration w/LID (mg/L)	Basin Plan Water Quality Objectives (narrative or mg/L)	Wasteload Allocations for MS4 Discharges into the Santa Clara River Reach 5 (mg/L)	Range of Observed[1] Concentrations in Santa Clara River Reach 5 (mg/L)	Average Wet Weather[2] Concentration at Station S29 (Days > 0.1")
TSS	33	Water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses	NA	32 – 51,200	1,060
Total Phosphorus	0.3	Waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses	NA	0.18 – 13.4	0.58
Total Nitrogen	2		NA	<0.04 – 46[6]	4.4
Nitrate-N + Nitrite-N	0.6	5	6.8[3]	0.5 – 4.8	0.9
Ammonia-N	0.2	2.2[4]	1.75[5]	<0.005 – 1.1	0.20
Chloride	14	100	100	3 - 121	43

[1] *Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3).*
[2] *Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.*
[3] *30-day average.*
[4] *4-day average, ELS present, 90th percentile pH and temperature pairing observed at USGS Monitoring Station 11108500.*
[5] *30-day average in Reach 5 below Valencia.*
[6] *Observed values for TKN (ammonia plus organic nitrogen).*

Comparison of the predicted runoff metal concentrations and the acute California Toxics Rule (CTR) criteria for dissolved copper, total lead, and dissolved zinc are shown in Table 6 below.

The comparison of the post-developed with LID condition to the benchmark CTR values shows that all of the trace metal concentrations are predicted to be below the benchmark water quality criteria. Predicted trace metals concentrations are within the range of observed concentrations in Santa Clara River Reach 5, except for dissolved zinc, which is slightly above the range of observed concentrations.

There is no CTR criterion for aluminum, although there is a National Ambient Water Quality Criteria (NAWQC) criterion (750 µg/L (acute) for a pH range of 6.5 to 9.0) in the form of acid soluble aluminum (USEPA, 1988). It is not possible to directly compare the predicted aluminum concentration to this criterion, as the available monitoring data used for modeling are for either dissolved aluminum or total aluminum. Acid soluble aluminum (which is operationally defined as the aluminum that passes through a 0.45 µm membrane filter after the sample has been acidified to a pH between 1.5 and 2.0 with nitric acid) represents the forms of aluminum toxic to aquatic life or that can be converted readily to toxic forms under natural conditions. The acid soluble measurement does not measure forms of aluminum that are included in total aluminum measurement, such as aluminum that is occluded in minerals, clays, and/or is strongly adsorbed to particulate matter, which are not toxic and are not likely to become toxic under natural conditions. The predicted mean total aluminum concentration (323 mg/L) is less than the NAWQC benchmark criterion for acid soluble aluminum, is predicted to decrease in the post-development condition, and is within the range of observed concentrations in Santa Clara River Reach 5.

Based on the comprehensive LID implementation strategy, the predicted decrease in runoff concentrations, and the comparison with benchmark objectives and instream concentrations, water quality impacts related to metals would be less-than-significant with the implementation of the proposed LID BMPs.

Table 6: Comparison of Predicted Trace Metal Concentrations for the Landmark Village Project Area with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Metal	Predicted Average Annual Concentration w/LID (µg/L)	California Toxics Rule Criteria[1] (µg/L)	Range of Observed[2] Concentrations in Santa Clara River Reach 5 (µg/L)	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
Dissolved Copper	10	32	3.3 – 22.6	7.3
Total Lead	4	260	0.6 – 40	18
Dissolved Zinc	60	250	3 – 37	19
Total Aluminum	323	N/A	131 – 19,650	5,500

[1] Hardness = 250 mg/L, based on minimum observed value at USGS Station 11108500. Lead criteria is for total recoverable lead. There is no CTR criterion for aluminum.

[2] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3).

[3] Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.

Assessment of Potential Project Impacts on Instream Concentrations

The potential for Project runoff to impact instream pollutant concentrations is a function of (1) the relative magnitudes of runoff volume and instream flow volume and (2) the relative magnitude of runoff concentrations and instream concentrations. The instream pollutant concentration with Project contributions can be calculated using a simple mass balance equation:

$$C_{IS} = \frac{V_O \times C_O + V_P \times C_P}{V_O + V_P}$$ Equation 1

Where:

C_{IS} = Instream Concentration with Project Runoff

V_O = Instream Volume Upstream of Project

C_O = Instream Concentration Upstream of Project

V_P = Volume of Runoff from Project Area

C_P = Concentration of Runoff from Project Area

This relationship can also be expressed as:

$$C_{IS} = \frac{L_O + L_P}{V_O + V_P}$$ Equation 2

Where:

L_O = Instream Constituent Load Upstream of Project

L_P = Constituent Load in Runoff from Project Area

Based on these relationships, two universal conditions can be identified under which a Project would not increase instream concentration:

- **Condition 1:** If the concentration of a constituent in Project runoff (C_P) is less than the concentration of the constituent instream (C_O), then discharges from the Project would result in a reduction of the instream concentration of that constituent; it would be not be possible for the Project's discharges to cause an increase in the instream concentration. Two extreme cases can be used to demonstrate this statement:

  - First, given that C_P is less than C_O, take the case where V_P is much less than V_O (e.g., the project size is small relative to the size of the watershed). In this case, the instream concentration would effectively equal C_O, although slightly less, indicating effectively no change in the instream concentration as a result of the project's discharges.
  - Given that C_P is less than C_O, take the case where V_P is much greater than V_O (the project size is very large relative to the size of the watershed). In this case, the instream concentration would effectively equal C_P, indicating that the project would reduce instream concentration because C_P is less than C_O.

- **Condition 2:** If the load of a constituent in Project runoff (L_P) decreases with development, but the volume of runoff from the Project increases (V_P), then the Project would be expected to result in a reduction of the instream concentration of that constituent regardless of instream volumes or concentrations. It would be impossible for the project to result in an increase in the instream concentration by reducing load but adding volume. In equation 2, this would effectively increase the numerator while reducing the denominator, which must cause the instream concentration to decrease.

The comparison of the post-developed with LID condition to the instream concentrations for the Landmark Village Project (**Error! Reference source not found.** and Table 6) shows that all pollutant concentrations, except dissolved zinc, are predicted to be below the average wet-weather instream concentration (Condition 1). On this basis, the Project would be expected to result in a reduction in the instream concentrations of these constituents.

Based on predicted changes in loads and volumes as a result of the Project with LID (Table 3), the average annual load of dissolved zinc is predicted to decrease with development, while runoff volumes are predicted to increase (Condition 2). On this basis, the Project would be expected to result in a reduction in the instream concentrations of dissolved zinc.

Cumulative Impact Assessment for LID Implementation

The *Landmark Village Water Quality Technical Report* evaluates cumulative impacts for the unincorporated area of Los Angeles County west of The Old Road to the Ventura County line. This geographic area includes the Newhall Ranch Specific Plan, Entrada, Legacy Village, and the remaining unbuilt portions of the Valencia Commerce Center. The LID Performance Standard described above will also be implemented by the other Specific Plan villages and the Entrada, Legacy Village, and Valencia Commerce Center projects.

The combined effect on modeled pollutant loads and concentrations of the NRSP, Entrada, Legacy Village, and the Valencia Commerce Center proposed projects are summarized in Tables 7 and 8 below, respectively. As shown in Table 7, when considered cumulatively, runoff

volumes and loads of Ammonia, dissolved copper, and chloride are predicted to increase from the NRSP, Entrada, Legacy Village, and Valencia Commerce Center projects, while pollutant loads are expected to decrease for TSS, total phosphorus, nitrate-N + nitrite-N, total nitrogen, total lead, dissolved zinc, and total aluminum. Pollutant concentrations from the combined projects are predicted to decrease for all modeled parameters (Table 8). Increases in pollutant loadings are not anticipated to be significant based on the fact that predicted pollutant concentrations are well below benchmark water quality standards and TMDL wasteload allocations and are within the range of observed concentrations in Santa Clara River Reach 5 (Table 9) with the exception of dissolved zinc. In the case of dissolved zinc, both the pollutant load and concentration are predicted to decrease with development.

Table 7: Predicted Average Annual Combined Runoff Volume and Pollutant Loads for the NRSP, Legacy Village, Entrada, and Valencia Commerce Center Projects

Modeled Parameter	Units	Development Condition			Change
		Existing	Developed with no BMPs	Developed with LID	
Volume	acre-ft	1,500	4,900	3,400	1,900
TSS	tons/yr	650	650	340	-310
Total Phosphorus	lbs/yr	5,500	4,300	1,800	-3,700
Nitrate-N + Nitrite-N	lbs/yr	16,000	13,700	6,100	-9,900
Ammonia-N	lbs/yr	1,900	7,500	2,100	200
Total Nitrogen	lbs/yr	25,000	44,000	19,000	-6,000
Chloride	tons/yr	43	135	88	45
Dissolved Copper	lbs/yr	32	130	55	23
Total Lead	lbs/yr	42	102	40	-2
Dissolved Zinc	lbs/yr	400	1,110	390	-10
Total Aluminum[1]	lbs/yr	6,300	10,400	5,400	-900

[1] *BMP effectiveness studies in the International Stormwater BMP database infrequently monitor aluminum; therefore, insufficient effluent data were available to model the removal effectiveness of treatment control BMPs for this water quality constituent. In order to estimate the reduction in total aluminum load and concentration, TSS removal was used as a surrogate.*

Table 8: Predicted Average Annual Combined Pollutant Concentrations for the NRSP, Legacy Village, Entrada, and Valencia Commerce Center Projects

Modeled Parameter	Units	Development Condition			Change
		Existing	Developed with no BMPs	Developed with LID	
TSS	mg/L	330	100	70	-260
Total Phosphorus	mg/L	1.4	0.3	0.2	-1.2
Nitrate-N + Nitrite-N	mg/L	4.0	1.0	0.7	-3.3
Ammonia-N	mg/L	0.5	0.6	0.2	-0.3
Total Nitrogen	mg/L	6	3	2	-4
Chloride	mg/L	22	20	19	-3
Dissolved Copper	µg/L	8	10	6	-2
Total Lead	µg/L	10	8	4	-6
Dissolved Zinc	µg/L	100	80	40	-60
Total Aluminum[1]	µg/L	1,580	780	590	-990

[1] *BMP effectiveness studies in the International Stormwater BMP database infrequently monitor aluminum; therefore, insufficient effluent data were available to model the removal effectiveness of treatment control BMPs for this water quality constituent. In order to estimate the reduction in total aluminum load and concentration, TSS removal was used as a surrogate.*

Table 9: Comparison of Predicted Pollutant Concentrations for the NRSP, Entrada, Legacy Village, and Valencia Commerce Center Projects with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Modeled Parameter	Units	Predicted Average Annual Concentration	TMDL/ LA Basin Plan Water Quality Objectives	California Toxics Rule Criteria[1]	Wasteload Allocations for MS4 Discharges into the Santa Clara River Reach 5	Range of Observed[2] Concentrations in Santa Clara River Reach 5	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
TSS	mg/L	70	Water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses	NA	NA	32 – 51,200	1,060
Total Phosphorus	mg/L	0.2	Waters shall not contain biostimulatory substances in concentrations that promote	NA	NA	0.18 – 13.4	0.58

Modeled Parameter	Units	Predicted Average Annual Concentration	TMDL/ LA Basin Plan Water Quality Objectives	California Toxics Rule Criteria[1]	Wasteload Allocations for MS4 Discharges into the Santa Clara River Reach 5	Range of Observed[2] Concentrations in Santa Clara River Reach 5	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
Total Nitrogen	mg/L	2	aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses	NA	NA	<0.04 – 46[7]	4.4
Nitrate-N + Nitrite-N	mg/L	0.7	5	NA	6.8[4]	0.5 – 4.8	0.9
Ammonia-N	mg/L	0.2	2.0[5]	NA	1.75[6]	<0.005 – 1.1	0.20
Chloride	mg/L	19	100	NA	100	3 - 121	43
Dissolved Copper	µg/L	6	NA	32	NA	3.3 – 22.6	7.3
Total Lead	µg/L	4	NA	260	NA	0.6 – 40	18
Dissolved Zinc	µg/L	40	NA	250	NA	3 – 37	19
Total Aluminum	µg/L	590	NA	NA	NA	131 – 19,650	5,500

[1] *Hardness = 250 mg/L, based on minimum observed value at USGS Station 11108500. Lead criteria is for total recoverable lead. There is no CTR criterion for aluminum.*
[2] *Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3).*
[3] *Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.*
[4] *30-day average.*
[5] *4-day average, ELS present, 90th percentile pH and temperature pairing observed at USGS Monitoring Station 11108500.*
[6] *30-day average in Reach 5 below Valencia.*
[7] *Observed values for TKN (ammonia plus organic nitrogen).*

As discussed above, the anticipated quality of effluent expected from the Landmark Village Project's PDFs will not contribute concentrations of pollutants of concern that would be expected to cause or contribute to a violation of the water quality standards in the Project's receiving waters. Therefore, the Project's incremental effects on surface water quality are not expected to be significant.

The Landmark Village Project's surface runoff water quality, after PDFs, both during construction and post-development, is predicted to comply with adopted regulatory requirements that are designed by the LARWQCB to assure that regional development does not adversely affect water quality, including MS4 Permit and SUSMP requirements; Construction General Permit requirements; General Dewatering Permit requirements; and benchmark Basin Plan water quality objectives, CTR criteria, and TMDLs. Any future urban development occurring in the

Santa Clara River watershed must also comply with these requirements. By extrapolating the results of the direct and cumulative impact analysis in this topical response, it can be predicted that analysis of other proposed development combined with existing conditions would have similar water quality results. Therefore, cumulative impacts on surface water quality of receiving waters from the Project and future urban development in the Santa Clara Watershed are addressed through compliance with the MS4 Permit and SUSMP requirements; Construction General Permit requirements; General Dewatering Permit requirements; and benchmark Basin Plan water quality objectives, CTR criteria, and TMDLs, which are intended to be protective of beneficial uses of the receiving waters. Based on compliance with these requirements designed to protect beneficial uses, cumulative water quality impacts are mitigated to a level that is less than significant.

Conclusion

None of the modeled pollutants of concern are expected to adversely affect water quality in surface waters, unreasonably affect present or anticipated beneficial uses of such waters, result in water quality less than that prescribed in the Basin Plan, or significantly impact receiving waters due to the implementation of the comprehensive LID Implementation Plan. Therefore, potential impacts from the Landmark Village Project on receiving water quality are not expected to be significant.

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LANDMARK VILLAGE LID PERFORMANCE STANDARD

LID project design features (PDFs) shall be selected and sized to retain the volume of stormwater runoff produced from a 0.75 inch storm event to reduce the percentage of Effective Impervious Area (EIA) to 5 percent or less of the total project area within the vesting tentative map project and associated off-site project area. Runoff from all EIA shall be treated with treatment control measures that are selected to address the pollutants of concern and are sized to capture and treat 80 percent of the average annual runoff volume.

Institutional, Commercial, Multi-Family Residential, Recreation, and Park Land Uses

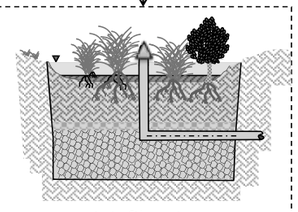
Attempt to retain ¾" storm

Assess infiltration feasibility

Infiltration Feasible
Category 1: Retain

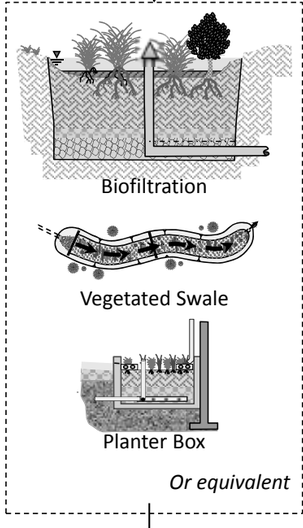
Or equivalent

Infiltration infeasible due to marginal infiltration rate, otherwise feasible
Category 2: Bioinfiltrate
Retain as feasible, then biofilter



Or equivalent

Infiltration infeasible due to hazards (geotechnical, etc.)
Category 3: Biofilter
Or route to regional facility



Or equivalent

Single Family Residential

Roofs, Patios and Walkways

Hydrologic Source Controls
Disconnect Roof Drains;
Disperse Impervious Areas to Retain 0.75" in Landscaped Areas

Remaining Area

Roads

EPA Green Streets Manual

Retain/biofilter as feasible per opportunities and constraints; Retain/biofilter remaining in Regional Facilities

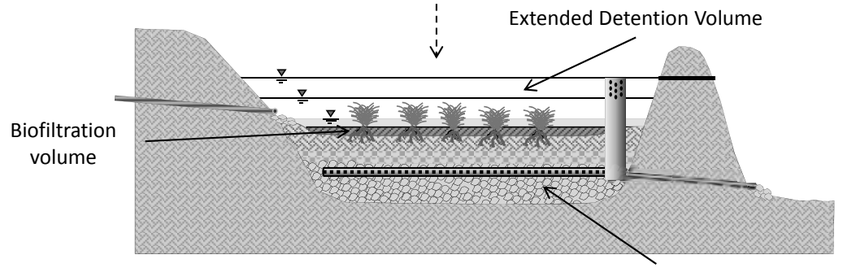
Allowable EIA

Treatment
80% capture of runoff

Media Filters
or equivalent

Underdrain Discharge (Category 2 and 3) and Bypassed Runoff

Sub-regional Bioinfiltration / Biofiltration Facility
Retain or biofilter to the extent feasible, plus provide additional volume required for extended detention for 80% capture of all runoff



Landmark Village LID Performance Standard
Landmark Village LID Water Quality Assessment

Geosyntec consultants

Figure 1

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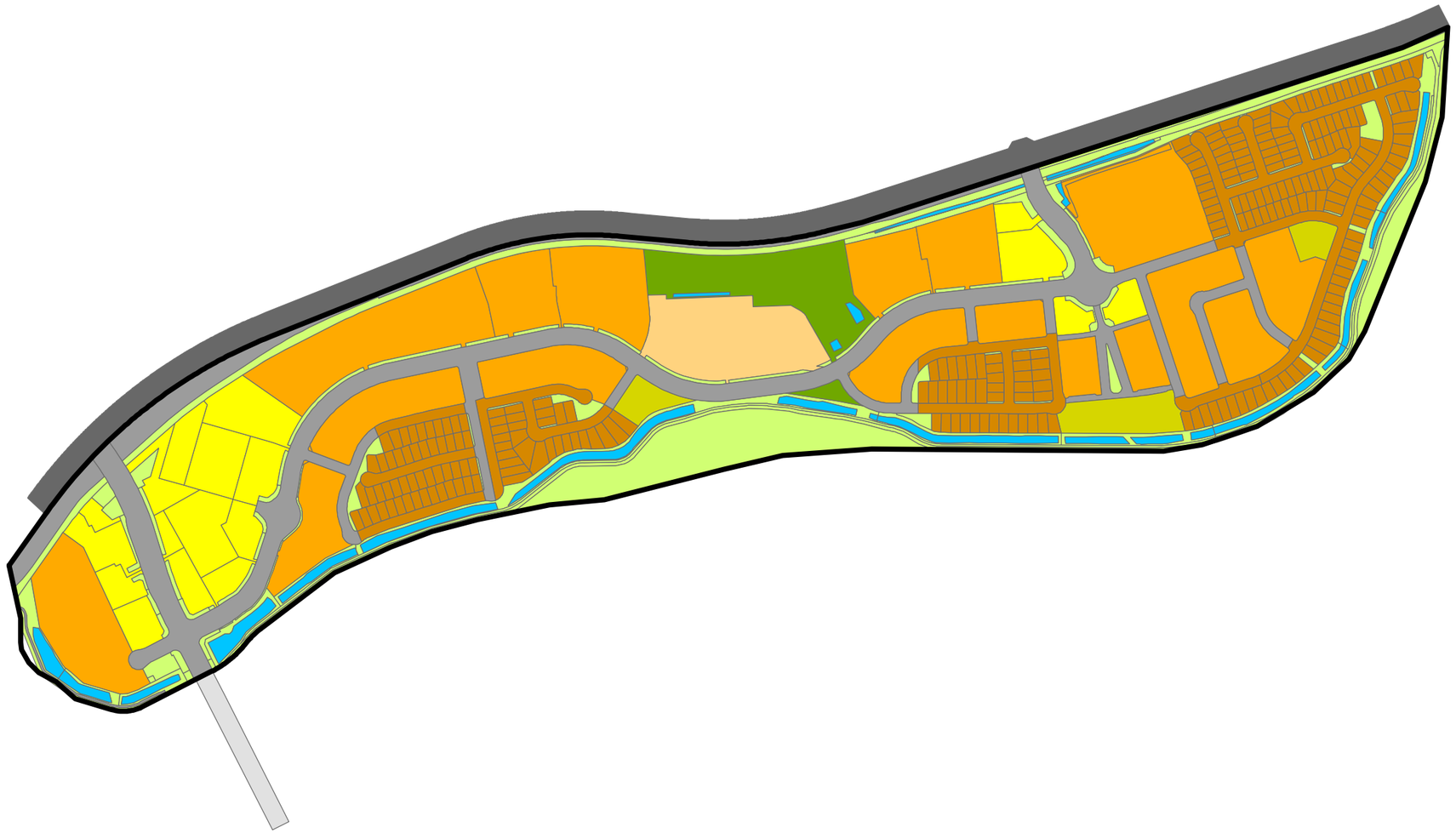

1,000 500 0 1,000 Feet

Legend

- Landmark Project Boundary

Model Land Use

- Single-Family
- Multi-Family
- Commercial
- Recreation
- School
- Park
- Open Space
- Water Quality Facility
- Road
- Off-site Bridge
- Off-site SR-126 Right of Way

Not shown: Off-site Water Tanks

Figure 2: Landmark Village Land Uses

May 2011

Geosyntec consultants

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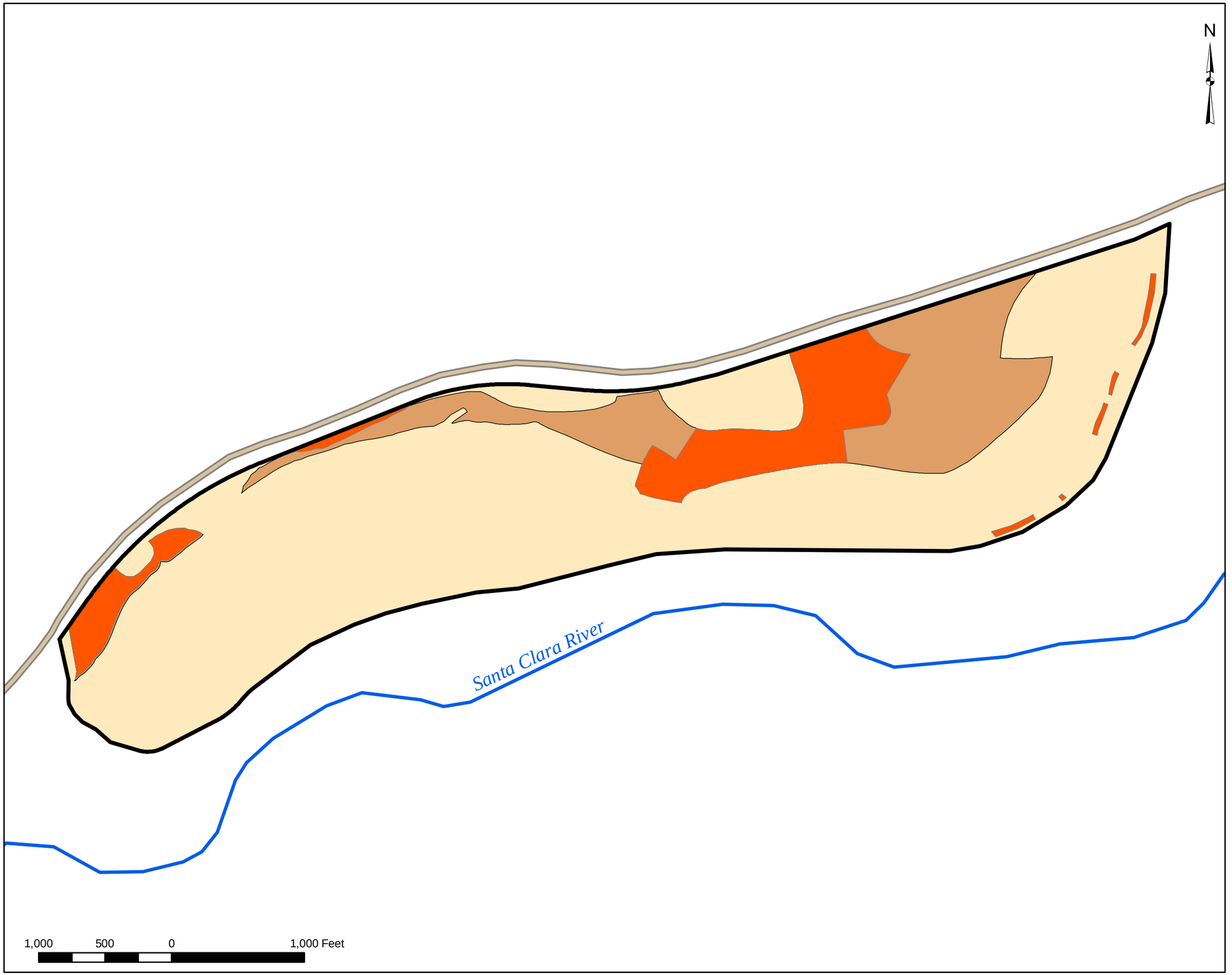
Prepared By: K. Havens



Legend

- Landmark Project Boundary
- Infiltration Feasible*
- Infiltration Feasible**
- Partial Infiltration Feasible

* Infiltration likely feasible directly from BMP bottom

** Infiltration likely feasible with supplemental dry well

Figure 3: Landmark Village Infiltration Feasibility Summary

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Prepared By: K. Havens

Note: Source: Alan E. Seward, 2010.

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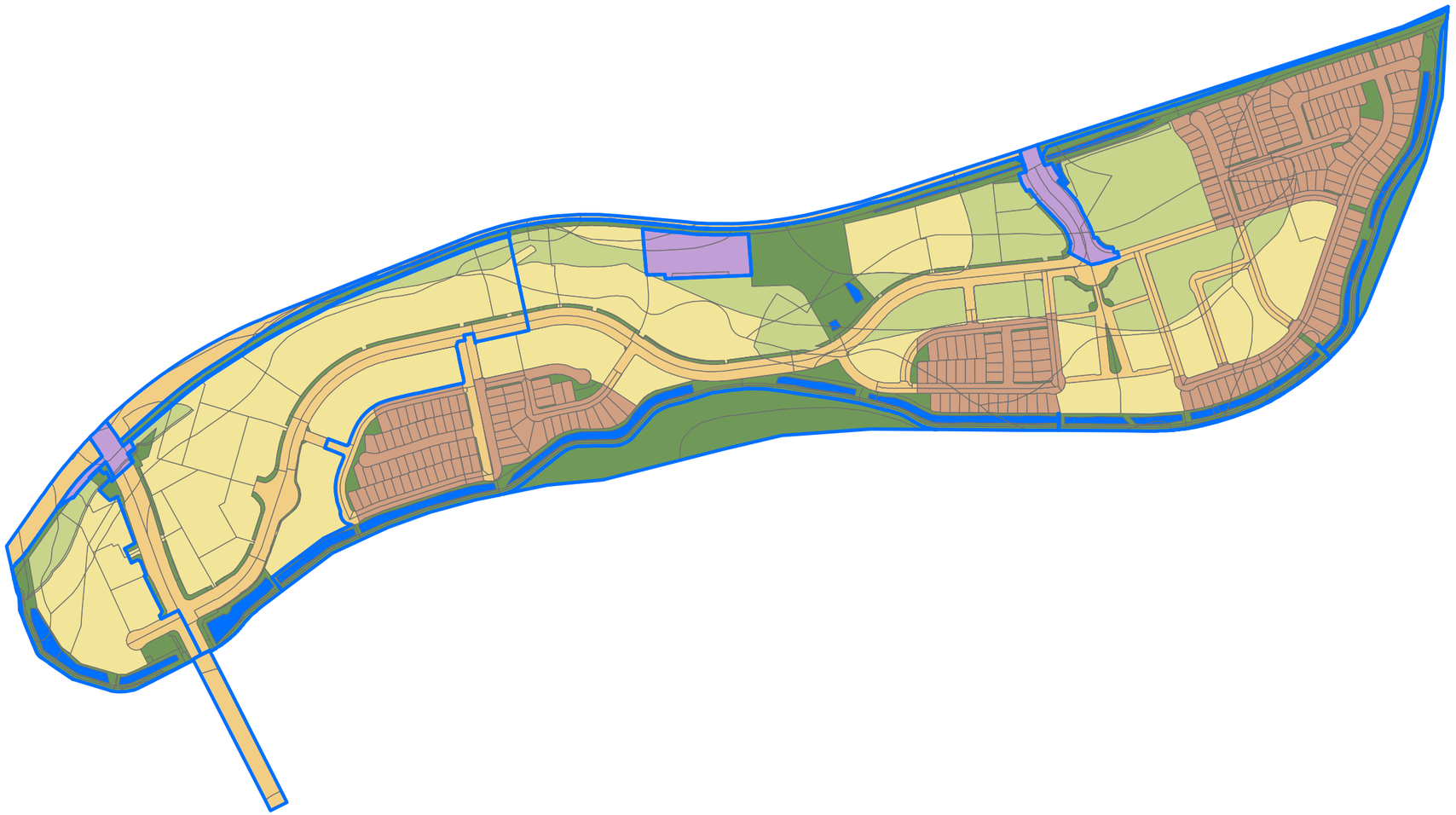

1,000 500 0 1,000 Feet

Legend

- Sub-regional and Treatment BMP Drainage Areas

Parcel-based Treatment Category:

- Open Space
- Category 1
- Category 2
- Green Streets
- Single-Family
- Treatment BMP
- Sub-regional Facility

Figure 4: Landmark Village Treatment Map

Landmark Village
LID BMP WQ Modeling Update

May 2011

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Prepared By: K. Havens

Attachment 1

LID Water Quality Modeling Methodology

Addendum to *Appendix B of the Landmark Village Water Quality Technical Report (February 2008)*

INTRODUCTION

The purpose of this attachment is to describe the changes to the water quality modeling methodology that have been made to quantify low impact development (LID) BMP implementation for the Landmark Village Project (Project) (i.e., the LID BMP Implementation Plan). Changes described in this attachment are discussed in comparison to the modeling methodology that is described in Appendix B of the Landmark Village Water Quality Technical Report (Landmark Village DEIR Appendix 4.3). This attachment addresses only the elements of the modeling methodology that have been updated, added, or clarified for the quantification of LID implementation for the Project.

This attachment is organized as follows:

- Section 1 provides an overview of the changes to the modeling methodology. This section also provides clarification of the rationales for the type of model employed for this analysis.
- Section 2 describes the updates made to model input parameters, as well as the updated approaches used to develop these input parameters.
- Section 3 describes the updates to the structure of the Monte Carlo model (i.e., the way the model is set up) to account for both on-parcel BMPs and sub-regional bioinfiltration/ biofiltration facilities. This section also provides an expanded discussion of the reliability of input parameters and assumptions.

1. MODEL OVERVIEW

1.1.1. Overview of Changes to Model Methodology

The overall modeling methodology has not changed substantively in comparison to the methodology described in Appendix B of the Landmark Village Water Quality Technical Report (WQTR). However, the structure of the model used to represent the Project (i.e., the way the model is set up) and some model inputs have changed to represent the LID Implementation Plan. Primary changes to the model structure and inputs include:

- Parcel-based BMPs were included in the model to account for volume reduction and treatment provided in parcel-based LID BMPs prior to draining to sub-regional bioinfiltration/ biofiltration facilities.

- The representations of sub-regional bioinfiltration facilities (previously called “Project Biobasins”) and the sub-regional biofiltration facility (previously called the “Project Extended Detention Basin”) were updated to reflect facility designs that include retention and biofiltration components which promote infiltration.

- BMP performance statistics were updated to support the simulation of the types of BMPs in the updated BMP plan using the latest version of the International BMP Database.

- The hydrology and hydraulic modeling approach used to develop hydrologic and hydraulic inputs to the Monte Carlo model (i.e., percent capture, percent volume reduction by storm event) was enhanced to more directly derive these estimates.

In addition, the model was updated to reflect the revised Project description and associated Project land use areas. The incorporation of these changes is described in further depth in Sections 2 and 3.

1.1.2. **Technical Basis for Modeling Methodology**

While the modeling methodology has not changed substantively, this section clarifies the technical basis and provides the rationale for the continued use of this methodology to evaluate Project stormwater quality impacts.

An empirical, pollutant loads model approach has been used to assess stormwater quality impacts associated with the proposed Project. This modeling approach was selected to meet the technical requirements of the water quality impact analysis based on an extensive review of available models and a review of the available datasets applicable to the Project.

A variety of modeling approaches are capable of meeting the technical requirements of this analysis. In general, models can be grouped into three categories:

- Stochastic (or probabilistic): this type of model utilizes observed statistical patterns to produce model estimates. This type of model generally relies on empirical observations, but does not necessarily ignore causal relationships.

- Deterministic (or mechanistic, physically-based): this type of model attempts to perfectly represent physical processes and mechanisms using closed form equations derived from physical phenomena. It is noted that because these models attempt to describe systems that are inherently complex and poorly defined, most deterministic models must rely in part on empirical observations to represent causal relationships.

- Hybrid: this type of model combines elements of stochastic and deterministic models to provide more reliable model estimates.

The modeling methodology used for the Project incorporates stochastic and empirical elements, and is therefore most accurately described as a hybrid approach. The approach uses an empirical, stochastic water quality estimation approach (Monte Carlo) to produce water quality and pollutant loading estimates. Inputs to this model are derived from empirical sources (Los Angeles County Land Use Monitoring Program and the ASCE International BMP Database) and deterministic modeling of hydrology and hydraulics (EPA SWMM4.4h). This approach makes use of robust land use and BMP monitoring datasets applicable to the project and incorporates important causal relationships in hydrologic and hydraulic response that can be reliably represented with deterministic methods. This approach is believed to be most appropriate to meet the technical requirements of the impact analysis for the Project-level analysis at the tract map scale.

The literature studies summarized below generally support the use of an empirically-based hybrid approach for the type of analysis required for the Project:

- Obropta et al. (2007) evaluated six deterministic models, three stochastic models, and three hybrid approaches. They concluded that *hybrid approaches show strong potential for reducing stormwater quality model prediction error and uncertainty* [improving the ability to assess] *best management practice design, land use change impact assessment* [and other applications].

- Charbeneau and Barrett (1998) evaluated different approaches for estimating stormwater pollutant loads based on a comparison of model results to observed land use monitoring data. They found that (1) the development of accurate physically-based models *remains a difficult and elusive goal*, and current understanding of processes *is not sufficient to accurately predict event loads*, (2) a simple empirical stochastic approach is generally as reliable or more reliable than more complicated mechanistic approaches, (3) the use of land use event mean concentrations (EMCs) is appropriate for planning purposes, (4) the land use EMC approach is most reliable when land use EMCs are used as a stochastic input parameter generated from a probabilistic distribution, and (5) stormwater volume is the single most important variable in predicting pollutant loads.

- The National Research Council's (NRC) 2008 report on *Urban Stormwater Management in the United States* generally supports these findings regarding the appropriate use of stormwater quality and quantity models.

Figure 1 provides an overview of the modeling methodology, with minor updates to support the updated BMP plan shown **in bold text**.

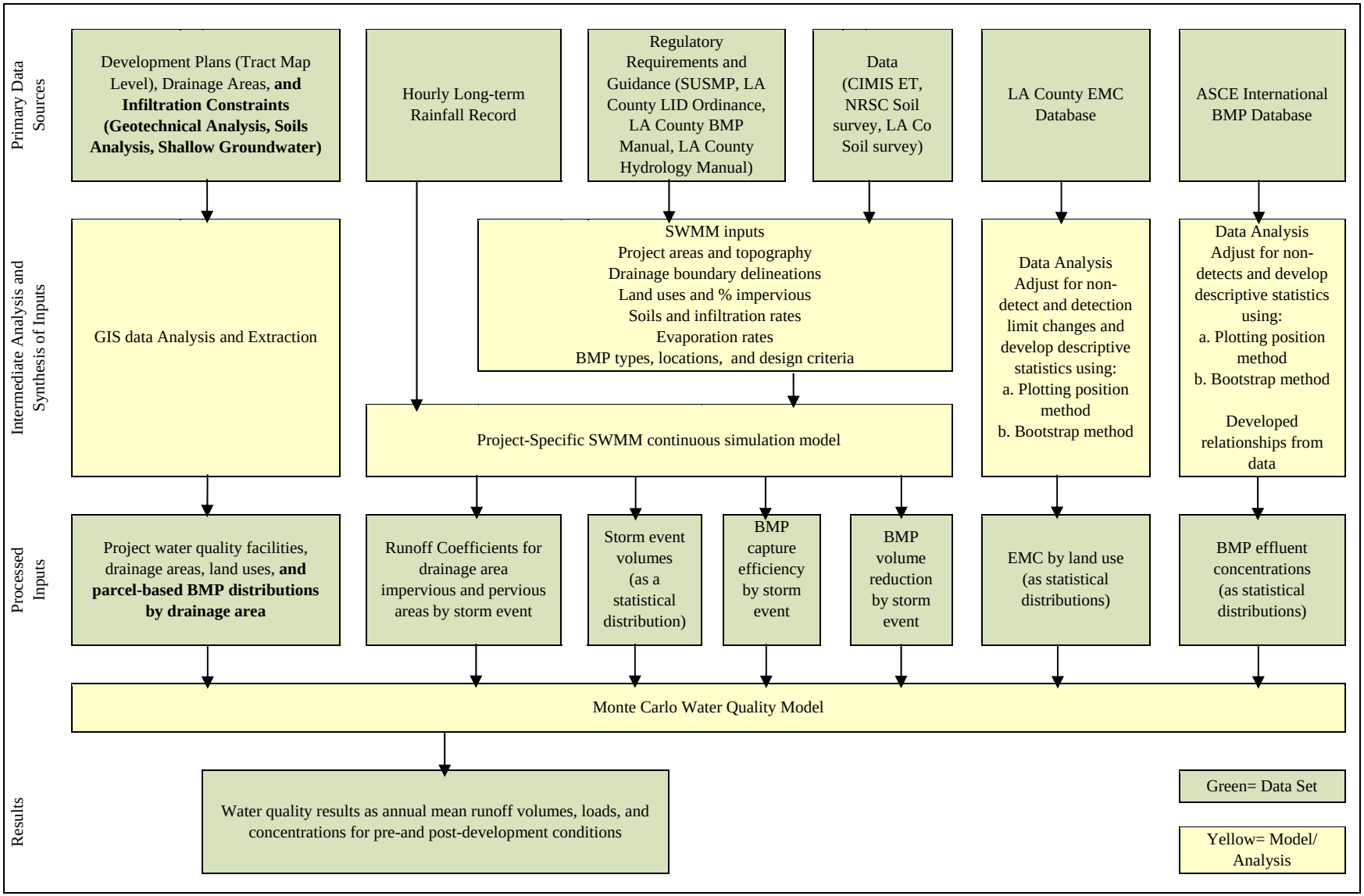

Figure 1: Overview of Updated Water Quality Analysis Methodology (Updates in Bold)

2. UPDATES TO MODEL INPUT PARAMETERS

2.1.1. Runoff Coefficients

As described in Appendix B of the Landmark Village WQTR, the Monte Carlo model uses runoff coefficients (derived from SWMM simulations of average drainage area soils conditions) as inputs to the modeling framework. Runoff coefficients for pervious portions of the Project area are based in part on the distribution of mapped soil properties in these areas. As a result of the change in Project development footprint, the distribution of soil properties changed slightly and triggered a re-analysis to develop runoff coefficients for pervious area. The updated soil distributions are presented in Table 1 below.

Table 1: SWMM Runoff Block Modeled Soils Distribution by WQ Drainage Area

	Sub-regional Bioinfiltration Facility		Sub-regional Biofiltration Facility[1]		On-site Treatment (Media Filter or Equivalent)		Off-site Swales		Off-site Swales	
Soil Group	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**
HSG A	43.3	21.3%	2.0	3.5%	0.8	10%	0	0%	0	0
HSG B	160.5	78.7%	54.7	96.5%	7.2	90%	8.2	100%	103.6	100%
Total	*203.8*		*56.7*		*8.0*		*8.2*		*103.6*	

[1] Includes 2.4 acres of off-site bridge.

The derivation of soil parameter assumptions for each hydrologic soil group (HSG) is summarized in Table 2. Suction head and initial moisture deficit (IMD) were estimated based on the soil texture class reported in the most Natural Resources Conservation Service (NRCS) Soil Survey (No. 675) with guidance on SWMM input parameter selection provided by James and James (2000). The saturated hydraulic conductivity (K_{sat}) was estimated based on evaluation of the ranges of undisturbed K_{sat} reported by the NRCS Soil Survey, the HSG reported by the NRCS Soil Survey with guidance on SWMM input parameter selection provided by James and James (2000), and the Los Angeles County Soil Type with infiltration characteristics described in the Los Angeles County Hydrology Manual (LA County, 2006). The preponderance of these soils data generally indicates that the project has relatively high infiltration capacity. Therefore, the assumed K_{sat} was based on the high end of the range of recommended SWMM inputs for A and B soils from James and James (2000).

Table 2: Green-Ampt Soil Parameters

Hydrologic Soil Group	Prevalent Soil Texture Class	Suction Head[1] (in)	IMD[1] (in/in)	Pre-Development K_{sat} (in/hr)	Post-Development K_{sat} (in/hr)
A	Loam	8	0.30	0.45	0.34
B	Loam	8	0.30	0.30	0.23

[1] Estimated based on texture class from Rawls, et al., (1983)

Based the soil distributions summarized in Table 1 and soil parameter assumptions summarized in Table 2, average pervious runoff coefficients were generated for each modeled drainage area to each type of sub-regional bioinfiltration/biofiltration BMP. Runoff coefficients are presented in Table 3 below. Runoff coefficients based on guidance from the Los Angeles County Hydrology Manual are also included in this table for reference (LACDPW, 2006).

Table 3: Runoff Coefficients by Water Quality Drainage Areas

WQ Drainage Area	Impervious Runoff Coefficient		Undeveloped Pervious Runoff Coefficient		Developed/Disturbed Pervious Runoff Coefficient	
	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)
Sub-regional Bioinfiltration Facilities	96.9	90	3	10	6	10
Sub-regional Biofiltration Facility	96.9	90	3	10	6	10
On-site Treatment (Media filters or equivalent)	96.9	90	3	10	6	10
On-site Biofilter Swales	96.9	90	3	10	6	10
Off-site Biofilter Swales (or equivalent)	96.9	90	3	10	6	10

2.1.2. **Revisions to Project Land Use**

Project land uses were determined from the Landmark Village Vesting Tentative Tract Map (VTTM #53108) and GIS analysis of this map (Psomas, April 2010) for the developed Project conditions, which have been revised for the Final Landmark Village EIR. In general, the assumptions regarding land use properties did not change, however, the revised land use plan

contains greater information about the types of roadways than was previously available, which required additional assumptions about the characteristic imperviousness and runoff quality from these areas, summarized below:

- For the purpose of analysis, roads adjacent to residential land uses were considered to be an integral part of single family detached land uses and assume all properties of this land use.

- Minor roads (private drives and access road) were grouped with major roads due to their proportionately small land area.

- Major roads were modeled using imperviousness and runoff quality associated with roads.

Developed conditions of the Project and associated off-site areas are summarized in Table 4.

Existing condition land uses have not changed since the previous model. Existing condition land uses can be found in Appendix B of Landmark Village WQTR.

Table 4: Developed Conditions Project and Off-site Land Uses

Land Use	Development Area (acres)		Impervious Fraction Modeled
	Project Site	Off-Site Impacts	
Modeled			
Commercial	27.3	8.0[2]	0.91
Multi-Family	82.9		0.74
Open Space	32.9		0.01
Park	10.1		0.10
Recreation	5.8		0.50
Road	41.6	98.0[3]	0.91
School	9.7		0.82
Single-Family[1]	53.9		0.42
Water Quality Facility	10.1		1.0
Not Modeled			
Open Space	18.2		NA
Total	*292.6*	*106.0*	

[1] Residential roads are included in the single-family land use. The Roads land use includes major and minor roads. Minor roads are modeled assuming the composite imperviousness and EMC of their adjacent land use types.
[2] Off-site water tanks are modeled as a commercial land use.
[3] Off-site roads consist of 2.4 acres of off-site bridge to the south and 95.6 acres of SR-126 right of way to the north. Impervious fraction of SR-126 ROW based on delineation of tentative improvement plans; not land use-based fraction.

2.1.3. Distribution of Parcel-based LID BMPs for Multi-Family, Commercial, Institutional, Recreation, and Park Land Uses

The LID BMP performance standard for the Landmark Village Project includes parcel-based LID BMPs in multi-family, commercial, institutional, recreation, and park land uses. There are three categories of parcel-based BMPs (Category 1, 2, and 3), the application of which depends on infiltration feasibility constraints associated with each land use parcel. Infiltration feasibility was screened as follows to determine the distribution of Category 1, 2, and 3 parcel-based BMPs:

- Infiltration feasibility constraints were evaluated by Seward (2010) to determine areas where infiltration is likely feasible. This analysis yielded a shapefile of infiltration feasibility constraints which displays locations where infiltration is likely feasible (Category 1) and where infiltration is likely partially feasible (Category 2). No areas on the Landmark project were identified where infiltration is not feasible in any level or would be hazardous (Category 3). Criteria associated with these distinctions are described in the *Landmark Village LID Supplement.*

- The land use program described in the Landmark Village VTTM (Psomas, April 2010) was converted to a GIS shapefile and was geospatially overlain with the shapefile of infiltration constraints to determine the location(s) and area of each type of parcel-based treatment within each drainage area.

The resulting distribution of parcel-based BMPs is shown in Table 5.

2.1.4. Distribution of Single Family Residential Hydrologic Source Controls

The LID BMP Implementation Plan includes hydrologic source controls (HSCs) in single family detached (SFD) land uses. For the purpose of modeling, it was assumed that rooftops, patios, and walkways would be routed to pervious areas capable of managing the runoff from at least a 0.75 inch storm event.

An analysis of typical development plans was conducted to determine the portion of the impervious area in the SFD residential land use that is made up of rooftops, patios, and walkways. Based on the project VTTM land use break-down, an area-weighted lot-size of 5,025 sq-ft plus 2,250 sq-ft of residential roadway was evaluated. Based on this analysis, it was found that on average approximately 26 percent of SFD land use area is anticipated to be made up of roofs, patios and walkways.

2.1.5. Design of Sub-regional Bioinfiltration/Biofiltration Facilities

The LID BMP Plan includes sub-regional water quality facilities, which are proposed to manage runoff from portions of the Project that are not addressed by parcel-based BMPs as well as bypass and treated discharge from parcel-based BMPs and SFD HSCs.

The areas draining directly to sub-regional facilities and the total tributary area to sub-regional facilities are provided in Table 5 below. Sub-regional bioinfiltration facilities are distributed around the Project site and have individual drainage areas. Because all sub-regional bioinfiltration facilities will be designed to the same design and performance standards, all area draining to sub-regional bioinfiltration facilities was considered to be part of on WQ drainage area for the purpose of modeling.

Table 5: Areas Draining to Parcel-based BMP Types within WQ Drainage Areas

	Category 1		Category 2		SFD HSC[1]		Directly to WQ Facility		*Total*	
WQ Drainage Area	**Area (Ac)**	**Imp (%)**	**Area (Ac)**	**Imp (%)**	**Area (Ac)**	**Imp (%)**	**Area (Ac)**	**Imp (%)**	**Area (Ac)**	**Imp (%)**
Sub-regional Bioinfiltration Facilities	36.1	77.7	50.0	73.4	28.1	50.0	89.6	41.1	203.8	56.8
Sub-regional Biofiltration Facility	2.8	78.0	36.3	81.9	0	0	17.6	71.3	56.7	78.4
On-Site Treatment (Media Filters or Equivalent)	No parcel-based BMPs						8.0	44.9	8.0	44.9
On-site Biofilter Swales (or equivalent)	No parcel-based BMPs						8.2	91.0	8.2	91.0
Off-site Biofilter Swales (or equivalent)	No parcel-based BMPs						103.6	91	103.6	91.0

1 - Includes single-family roofs, patios and sidewalks, draining to a pervious area with an equivalent square-footage.

2.1.6. **Updated BMP Performance Parameters**

As in the previous modeling methodology, the performance of project BMPs is estimated as a function of three factors: (1) the fraction of stormwater runoff receiving treatment (often referred to as percent of runoff captured, or simply percent capture); (2) the pollutant removal achieved in the BMP by virtue of infiltration and/or evapotranspiration (generically referred to as volume reduction); and (3) the pollutant removal achieved in the BMP by virtue of improved water quality. The performance parameters associated with these factors have been updated to reflect the LID BMP Plan as described in the sections below.

2.1.7. **BMP Capture Efficiency and Volume Reduction**

The Monte Carlo model utilizes event-by-event estimates of BMP capture efficiencies and volume reduction to describe the hydrologic and hydraulic performance of BMPs. These inputs were developed using SWMM simulations. While this approach has not fundamentally changed, slight changes were required to accommodate the LID BMP Plan including (a) the simulation of parcel-based BMPs that are “nested” within the drainage area of sub-regional facilities, (b) the

simulation of SFD HSCs that provide volume reduction from SFD land uses, and (c) the simulation of sub-regional bioinfiltration/ biofiltration facilities that combine infiltration and biofiltration elements in different proportions depending on infiltration feasibility in the location of the facility. The approaches for developing capture efficiency and volume reduction inputs for the Monte Carlo model for each of these BMP types are described in the sections below. These approaches make use of the SWMM Runoff block (hydrologic simulation module) and the SWMM Storage/Treatment block (hydraulic simulation module), both operated in continuous simulation mode for a period of 40 years.

2.1.7.1. Parcel-based BMPs

Estimates of capture efficiency and volume reductions achieved by parcel-based BMPs were developed based on hydraulic representations of parcel-based BMPs in EPA SWMM4.4h (Storage/Treatment block), with spatially-averaged tributary catchments (Runoff block). A hypothetical spatially-averaged catchment representation was used because exact drainage areas and imperviousness for each parcel-based BMP are not available at this level of analysis (i.e., Tier 2, the tract map project scale). The hypothetical spatially-averaged catchment was assigned an area of one acre and an impervious fraction representative of the composite imperviousness of Project areas draining to parcel-based BMPs. This catchment was simulated in the SWMM Runoff block to produce a characteristic runoff hydrograph, which was routed through each type of parcel-based BMPs using the SWMM Storage/Treatment block. The reliability of the spatially-averaged catchment approach is discussed in Section 3.2.

The hydraulic representation of each type of parcel-based BMP was developed in the SWMM Storage/Treatment block based on a standard BMP profile formulated to result in the maximum feasible infiltration of the 0.75 inch design storm for each infiltration constraint condition. The standard profiles are primarily dependent on the design infiltration rate of underlying soil for each of the constraints categories. Based on an assessment of likely infiltration rates and allowable infiltration volumes (Seward, 2010), the design infiltration rates were selected as follows.

- **Category 1** areas are located in areas identified as having a natural, undisturbed infiltration rate of greater than 0.5 inches per hour and the potential to use direct infiltration or dry wells to infiltration. Direct infiltration was assumed to be feasible in areas with depth of fill less than 10 feet. Dry wells were assumed to be feasible in areas where the depth from the bottom of fill to seasonally-high groundwater is greater 10 feet. The design infiltration rate for this category of parcel-based BMP was selected by applying a reduction factor of 25 percent to the low end of estimated infiltration rate. The result is a design infiltration rate of **0.375 inches per hour**.
- **Category 2** areas are generally located in areas with natural, undisturbed infiltration rate of less than 0.5 inches per hour (Seward, 2010) and/or where depth of fill or separation from the bottom of fill to groundwater would not permit full infiltration of the design

storm volume. The design infiltration rate for this category of parcel-based BMP was selected by applying a reduction factor of 50 percent to the low end of estimated infiltration rate. The result is a design infiltration rate of **0.25 inches per hour**. This assumption considers physical limitations of infiltration into compacted and low permeability soils as well as hazards associated with introduction of excess water into fill structures.

Based on these design infiltration rates and the design goals for parcel-based BMPs described above, the geometric inputs to the SWMM hydraulic representations of parcel-based BMP are described in the Table 6 below.

Table 6: SWMM Hydraulic Representation of Parcel-based BMPs

Parameter	Units	Parcel-based BMP Categories	
		Category 1	Category 2
Surface Ponding Depth below Overflow	ft	1.0	0.5
Media Depth	ft	1.5	1.5
Design K_{sat} of Amended Media	in/hr	2.0	2.0
Design K_{sat} of Underlying Soil	in/hr	0.375	0.25
Thickness of Gravel Layer	ft	0	1.5
Height of Underdrain Invert Elevation above Bottom of BMP	ft	None	1.5
Depth of Retention Storage[1]	inches	18.3	9.0
BMP Footprint as Fraction of Impervious Area	ac/ac	3.1%	2.9%
Average Annual Capture Efficiency (Percent Capture)	-	53%	80%
Average Annual Volume Reduction of Captured Water (Percent Volume Reduction)	-	100%	41%
Average Annual Reduction in Runoff Volume	-	53%	33%

[1] Retention storage depth is determined based on the equivalent depths of volume retained in ponding, media, and gravel (i.e. the full storage volume of Category 1 and, for Category 2, volume below underdrain), as well as additional retention storage in media.

The storm-by-storm capture efficiency and volume reduction estimated from the parcel-based BMP simulations was extracted from SWMM model output and used to represent the hydraulic performance of these BMPs in the Monte Carlo model.

2.1.7.2. SFD Hydrologic Source Controls

The effect of HSCs was simulated by routing runoff from impervious areas to pervious areas within the SWMM Runoff block (hydrologic simulation module) and tabulating the combined runoff coefficient from this area for each storm event. For the purpose of analysis, it was assumed that impervious areas would be routed over an equal amount of pervious area with

properties modified to represent amended soils in the areas receiving runoff. Table 7 provides the model parameters that were used to represent SFD HSCs.

Table 7: SWMM Model Representation of Hydrologic Source Controls

Parameter	Units	Assumption	Basis of Assumption
Impervious to Pervious Ratio	ft	1:1	Based on typical available landscape area per tributary area, or equivalent HSC
Slope of Pervious Area	ft/ft	0.05	See Appendix B
Depression Storage of Pervious Area	inches	0.5	Based on soil amendments to 4 inch depth improving soil moisture storage capacity by 0.125 inches per inch; actual design of HSCs may vary
Manning's Surface Roughness of Pervious Area	-	0.25	See Appendix B (tables)
K_{sat} of Pervious Area	in/hr	Based on drainage area average developed K_{sat}	See Appendix B (tables)
Saturated Hydraulic Conductivity of Pervious Area (K_{sat})	in/hr	Varies based on soil type	See Table 2
Suction Head of Pervious Area	inches	8.0	See Table 2
Initial Moisture Deficit of Pervious Area	in/in	0.3	See Table 2
Runoff coefficient of impervious plus pervious area	-	13.3	Modeled in SWMM. Takes into account infiltration of runoff from impervious area in pervious area.

The effect of HSCs was accounted in the Monte Carlo model by modifying the runoff coefficient of the areas being disconnected and receiving disconnection. The runoff coefficient of this area was tabulated from SWMM output for each storm event.

2.1.7.3. Sub-regional Bioinfiltration/Biofiltration Facilities

The hydraulic performance of each sub-regional bioinfiltration/biofiltration facility is dependent on characteristics of the tributary drainage area (including the amount of parcel-based BMPs and HSCs provided in the tributary drainage area), the volume of the facility, the underlying design infiltration rate, and the outlet control configuration. Therefore, to evaluate the capture efficiency and volume reduction performance of sub-regional facilities, drainage area hydrologic representations and facility hydraulic representations were developed for each facility.

Drainage Area Representation for Sub-regional Facilities

The drainage area representation used to evaluate sub-regional facilities was developed using the same approach described in Landmark Village WQTR Appendix B, with the exception that the effects of "nested" parcel-based BMPs and SFD HSCs were approximated by embedding

hydrologic elements in the drainage area representation to represent these BMPs (i.e., hydrologic BMP representations).

To approximately account for the effects of parcel-based BMPs in each sub-regional facility drainage area, “hydrologic representations” of parcel-based BMPs were used. These representations do not account for detailed hydraulic routing, but generally account for the effect of parcel-based BMPs on the overall volumetric response from the drainage area. These representations included increasing the depression storage of selected pervious and impervious areas, and routing impervious area runoff to these “sump” areas based on the distribution of Category 1 and 2 LID BMPs in each WQ drainage area described in Table 5.

To ensure that this representation provides a reasonably accurate approximation of the effects of parcel-based BMPs, the volume reductions resulting from this hydrologic representation were compared to the volume reductions resulting from the more detailed hydraulic representations described in Section 2.1.7.1. The pervious or impervious depression storage values used in the hydrologic representations were adjusted such that the average annual volume reductions due to depression storage losses (i.e., hydrologic representations of parcel-based BMPs) were equivalent to the average annual volume reductions achieved in the hydraulic representations of parcel-based BMP. The adjusted impervious or pervious depression storage depths used for the drainage area hydrologic representations of parcel-based BMP are reported in Table 8 below. The reliability of this approach is discussed in Section 3.2.

Table 8: SWMM Hydrologic Model Representation of Parcel-based BMPs

SWMM Runoff Parameters	Units	Parcel-based LID BMP Type	
		Category 1	Category 2
Depression storage, pervious	inches	21	10
Depression storage, impervious	inches	NA	NA
Imperviousness	%	0	0
Infiltration Rate	in/hr	0.375	0.15
Average Annual Reduction in Runoff Volume from Hydrologic Representation	-	53%	33%
Average Annual Reduction in Runoff Volume from Hydraulic Representation (See Table 6)	-	53%	33%

The selected footprint areas of the parcel-based BMPs for these hydrologic representations were determined by scaling the footprint areas generated from the hydraulic parcel-based BMP representations based on the impervious fraction of the drainage area.

To represent the hydrologic effects of SFD HSCs in the sub-regional facility drainage area representation, the portions of the drainage area attributed to SFD rooftops, patios, and walkways

was routed over pervious landscape areas in a one-to-one ratio. Parameters used to represent this disconnection scenario are reported in Table 7.

Hydraulic Representation of Sub-regional facilities
Sub-regional bioinfiltration/biofiltration facilities were represented in the SWMM Storage/Treatment block based on the proposed designs of these facilities.

Designs were developed by first estimating the infiltration rate of soils underlying the proposed facility location and identifying any other constraints on infiltration (Table 9).

Table 9: Sub-regional Facility Design Infiltration Rates

Facility Type	Assumed Design Infiltration Rate, inches per hour	Basis for Assumption
Sub-regional Bioinfiltration Facilities	0.25	Seward screening shows areas have infiltration rates greater than 0.5 in/hr. Factor of safety of 2 applied.
Sub-regional Biofiltration Facility	0.25	Seward screening shows areas have infiltration rates greater than 0.5 in/hr. Factor of safety of 2 applied.
On-site Treatment (Media filter of equivalent)	No infiltration assumed	Are may be treated by a variety of proprietary systems that do not promote infiltration.
On-site and Off-site Biofilter Swales	NA - Infiltration not modeled explicitly	Because designs of biofilter swales have not been developed for all areas, estimates of volume reductions expected in swales were derived from analysis of the International BMP Database.

A standard profile for each sub-regional facility was then developed based on the portion of the facility volume that can be dedicated to infiltration and the portion of the facility volume that is treated and released. This is a function of the design infiltration rate of soil under the facility.

Finally, the geometry of the basins was determined via iterative model runs to meet the following criteria:

- Surface storage draws down in less than or equal to 48 hours (subsurface storage in the pore spaces of gravel and suction storage in media pores may persist for longer than 48 hours as this storage does not pose a risk related to vector control or habitat creation).

- The facility captures and retains or treats runoff volumes such that less than 20 percent of the baseline drainage area runoff volume “bypasses” the facility (i.e., is routed around the facility or flows through the facility without significant treatment). The baseline drainage area runoff volume is defined as the volume that would occur without parcel-based BMPs or SFD HSCs. Limiting the sub-regional facility bypass to 20 percent of the baseline volume ensures that the Project performance standard of 80 percent capture is achieved

on a drainage area basis, including the volume reduction effect of BMPs that are nested in the drainage area plus the volume reduction and treatment provided in the downstream sub-regional facility.

After an iterative solution was found that meets these criteria, the capture efficiency and volume reduction were tabulated for each storm event by post-processing SWMM model output. The estimated capture efficiency and volume reduction on a storm-by-storm basis were used to describe hydraulic performance of sub-regional facilities in the Monte Carlo model.

Sub-regional infiltration/ biofiltration facility type and geometries are listed in Table 10.

Table 10: Sub-regional Bioinfiltration/Biofiltration Facility Geometry

Parameter	**Units**	**Sub-regional Bioinfiltration Facilities**	**Sub-regional Biofiltration Facility**
Facility Type	--	Shallow vegetated basins with vegetated media filtration and gravel sump below underdrain	Basin-type BMP with extended detention, biofiltration, and incidental infiltration
Facility Volume	ac-ft	Varies by facility	4.3
Surface Ponding Depth below Overflow	ft	1.5	6
Surface Drawdown Time	hours	9	48
Assumed Design Saturated Hydraulic Conductivity (K_{sat}) of Underlying Soil	in/hr	0.25	0.25
Assumed Biofiltration Media Thickness	ft	2 or greater	1.5 or greater
Water Equivalent Retention Depth below Underdrain	ft	0.5	None
Subsurface Drawdown Time	hours	24	NA

Table 12 reports long-term hydrologic performance of sub-regional facilities (capture efficiency and volume reduction) as well as the overall drainage area capture and volume reduction inclusive of volume reductions achieved in nested parcel-based BMPs, hydrologic source controls, and sub-regional facilities.

Table 11: Sub-regional Facility Hydraulic Performance and Drainage Area Total Performance

WQ Drainage Area	Total Tributary Area	Composite % Imp	Sub-Regional Facility Capture Efficiency of Runoff Volume	Sub-Regional Facility Volume Reduction of Captured Water	Parcel-based Volume Reduction Upstream of Sub-regional Facility	Drainage Area Total Capture Efficiency	Drainage Area Total Volume Reduction
Sub-regional Bioinfiltration Facilities	203.8	57%	71%	33%	32%	80%	48%
Sub-regional Biofiltration Facility[1]	56.7	78%	74%	16%	24%	80%	34%
On-site Treatment (Media filter or equivalent)	8.0	45%	80%	0%	0%	80%	0%
Biofilter Swales (on-site)	8.2	91%	80%	20%	0%	80%	20%
Biofilter Swales (off-site)	103.6	91%	80%	20%	0%	80%	20%

[1] Includes 2.4 acres of off-site roadway (bridge) that drains to this BMP.

2.1.8. **BMP Pollutant Removal**

The Monte Carlo model characterizes BMP pollutant removal as a function of BMP effluent quality (statistical distributions and irreducible concentration) derived from analysis of the International BMP Database (www.bmpdatabase.org). To support the updated BMP plan, the latest version of the BMP Database (obtained 1/13/2011) was queried and analyzed to produce effluent quality distributions characteristic of the types of BMPs included in the updated BMP plan. Project BMP types were matched to the most representative category of BMP in the BMP Database for the purpose of modeling (Table 12).

Table 12: BMP Effluent Quality Performance Parameters

BMP Type	Facility Type	BMP has Treated Effluent?	BMP Database Category for Effluent Quality
Sub-regional Bioinfiltration Facilities	Infiltration and Biofiltration	Y	Media Filter
Sub-regional Biofiltration Facility	Extended detention, biofiltration and incidental infiltration	Y	Media filter plus detention basin treatment train
Media Filters or equivalent	Flow-based BMPs incorporating media filtration or equivalent treatment mechanisms	Y	Media Filter
Biofilter Swales (on- and off-site)	Biofilter Swales	Y	Biofilter
Parcel-based Category 1 LID BMPs	Infiltration	N	NA
Parcel-based Category 2 LID BMPs	Infiltration and Biofiltration	Y	Media Filter
SFD HSC	Infiltration and ET	N	NA

NA – BMP does not have treated effluent.

Table 13 summarizes the number of data points (individual storm events) and percent non-detects for the pollutants and BMP types of interest for which sufficient data were available. Table 14 summarizes the log-normal statistics that were used in the water quality model, and Table 15 summarizes arithmetic descriptive statistics for those data sets. Table 16 summarizes the irreducible effluent concentration estimates used by for water quality modeling of the proposed condition. A full description of the statistical analysis methods and assumptions used to generate BMP descriptive statistics is contained in Appendix B of the Landmark Village WQTR Appendix B (February 2008). Note that because of a paucity of data in the BMP Database for some pollutants, no treatment was assumed for nitrite (NO_2), total aluminum, and chloride, so these constituents are not included on the following summary charts even though they were included in the model. Load reductions are still possible for these pollutants via volume reduction provided in BMPs.

Table 13: Summary of Number of Data Points and Percent Non-Detects for BMP Effluent Concentration Data from the International BMP Database

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
Media Filter	Count	294	292	135	99	263	186	254	185
	% ND	9%	8%	39%	3%	3%	7%	30%	21%
Detention Basin	Count	509	258	85	93	178	173	197	174
	% ND	0%	3%	7%	12%	4%	32%	46%	9%
Biofilter (Swale)	Count	461	547	361	312	499	255	455	255
	% ND	2%	2%	15%	0%	0%	3%	35%	10%

Table 14: International BMP Database Lognormal Statistics of BMP Effluent Concentrations

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
Media Filter	Mean	2.54	-2.30	-2.59	-1.10	-0.24	1.46	0.89	3.07
	St. Dev	1.24	1.06	1.27	1.15	1.01	1.15	1.26	1.12
Detention Basin	Mean	2.56	-1.75	-2.25	-2.09	-0.08	1.36	1.88	2.81
	St. Dev	1.25	1.23	0.92	1.66	1.39	0.95	1.19	1.03
Media Filters plus Detention Basin Treatment Train[1]	Mean	2.54	-2.30	-2.59	-2.09	-0.24	1.46	0.89	3.07
	St. Dev	1.24	1.06	1.27	1.66	1.01	1.15	1.26	1.12
Biofilter (Swale)	Mean	2.66	-1.78	-2.44	-1.68	-0.30	1.81	0.91	2.76
	St. Dev	1.06	1.17	1.37	1.01	0.94	0.69	1.36	1.05

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

Table 15: International BMP Database Arithmetic Estimates of BMP Effluent Concentrations

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
	units	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*ug/L*	*ug/L*	*ug/L*
Media Filter	Mean	27.54	0.18	0.17	0.65	1.32	8.28	5.42	40.25
	St. Dev	52.86	0.26	0.34	1.08	1.77	13.65	10.75	63.56
Detention Basin	Mean	28.29	0.37	0.16	0.49	2.43	6.10	13.30	28.12
	St. Dev	54.70	0.69	0.19	1.90	5.95	7.41	23.40	38.68
Media Filters plus Detention Basin Treatment Train[1]	Mean	27.54	0.18	0.17	0.49	1.32	8.28	5.42	40.25
	St. Dev	52.86	0.26	0.34	1.90	1.77	13.65	10.75	63.56
Biofilter (Swale)	Mean	25.24	0.34	0.22	0.31	1.16	7.81	6.26	27.44
	St. Dev	36.53	0.58	0.52	0.41	1.39	6.14	14.39	38.72

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

Table 16: International BMP Database Arithmetic Irreducible Effluent Concentration Estimates

BMP	TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
	mg/L	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*ug/L*	*ug/L*	*ug/L*
Media Filter	1.49	0.03	0.01	0.01	0.24	0.99	0.20	1.35
Detention Basin	2.99	0.09	0.34	0.02	3.78	15.16	0.27	78.83
Media Filters plus Detention Basin Treatment Train[1]	1.49	0.03	0.01	0.02	0.24	0.99	0.20	1.35
Biofilter (Swale)	2.02	0.07	0.01	0.66	0.19	2.03	0.32	4.95

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

3. MONTE CARLO MODEL

3.1.1. Updates to Model Methodology

The Monte Carlo model framework used to simulated the LID BMP Implementation Plan is identical to that described in Appendix B of the Landmark Village WQTR, however, the model structure (i.e., the way the model is set up) has been modified somewhat to account for the volume and pollutant load reductions achieved through “nested” parcel-based BMPs and SFD HSCs upstream of sub-regional facilities. Accounting for these nested BMP requires another “loop” of pollutant load generation, removal and routing algorithms to be implemented in the model within each sub-regional facility drainage area for each simulated event. An overview of the revised model structure to account for “nested” BMPs is illustrated in Figure 2.

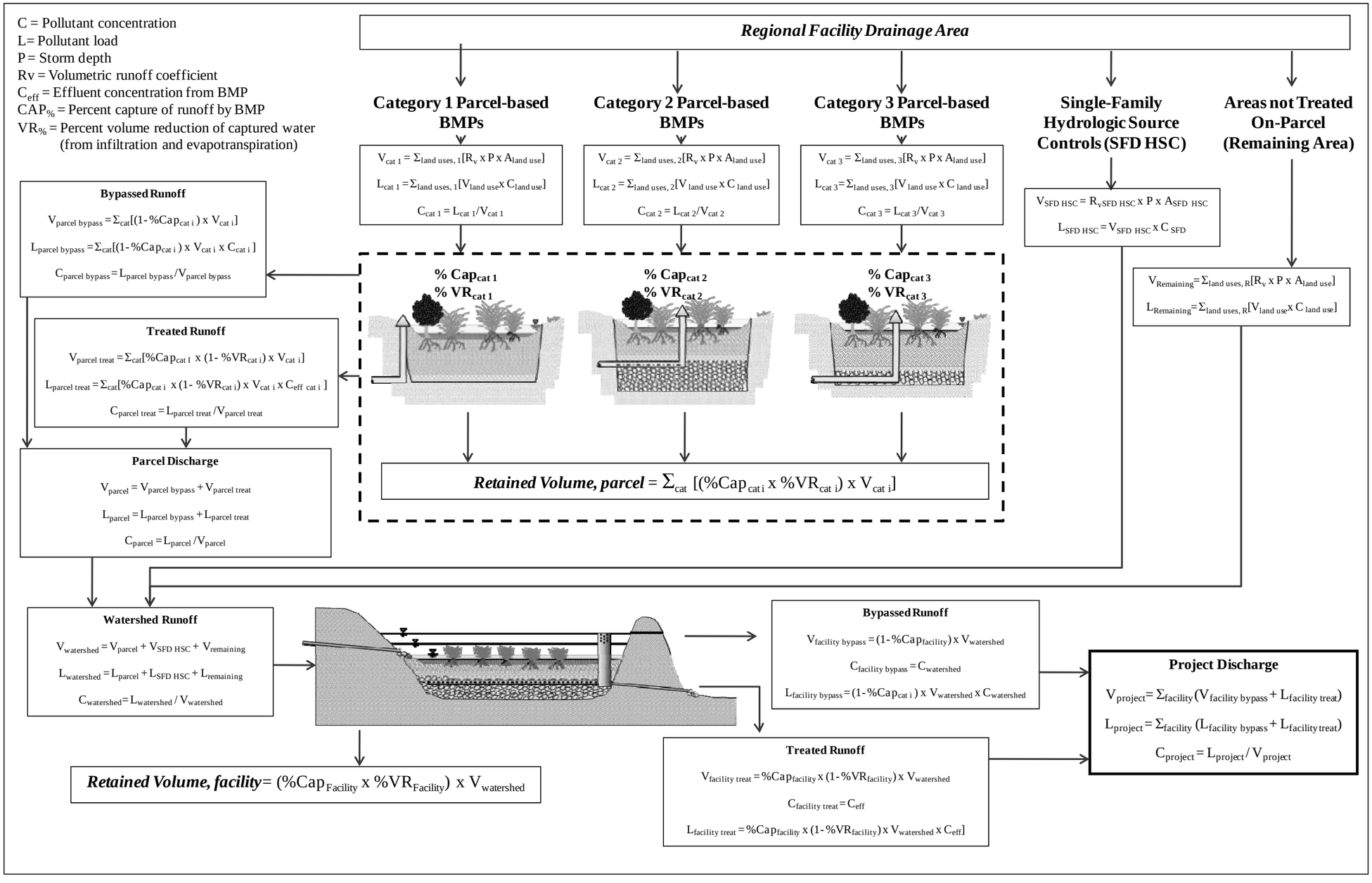

Figure 2: Monte Carlo Model Schematic with Nested LID BMPs and Hydrologic Source Controls (Generalized Schematic)

3.2.**Model Parameter Reliability and Assumptions**

This section discusses the reliability of new or revised model parameters and assumptions necessary to support the LID BMP Plan.

Drainage Area Runoff Coefficients and Hydrologic Parameter Sensitivity

The estimation of runoff coefficients is highly dependent on soil properties (i.e., infiltration potential) and less dependent on parameters such as evapotranspiration (ET) rates, slopes, and surface roughness. Soil properties are estimated as accurately as possible from available soils data, incorporating the latest soil survey conducted by the USDA NRCS as well as locally-developed infiltration relationships provided in the Los Angeles County Hydrology Manual (LACDPW, 2006). The resultant estimates of runoff coefficients that may somewhat overestimate or underestimate stormwater runoff.

Table 17 provides a comparison of assumed project runoff coefficients (developed from SWMM modeling) to applicable references.

Table 17: Comparison of project runoff coefficients to applicable references

Drainage Area Imperviousness	**Project Runoff Coefficient Assumptions** (varies by drainage area; range provided)	**LA County Hydrology Manual** (Minimum Cu = 0.1)	**Ventura County Manual,** Silty Clay Soils (Soil Types 2 or 3)	**ASCE/WEF Manual of Practice 23/87** (3rd order polynomial)
Reference	Table 3	LACDPW, 2006	Ventura County, 2010	ASCE/WEF, 1998
90% impervious, developed	0.88	0.82	0.87	0.73
60% impervious, developed	0.6 - 0.62	0.58	0.61	0.41
30% impervious, developed	0.33 - 0.36	0.34	0.36	0.23
1% impervious, undeveloped	0.03 - 0.07	0.11	0.11	0.05

Based on the comparison provided in Table 17, the assumed developed condition runoff coefficients are reasonably consistent with applicable references, although generally somewhat high. Assumed undeveloped condition runoff coefficients are generally lower than applicable references. The combined effect of these trends results in a somewhat higher estimate of impacts associated with the Project and somewhat higher estimate of absolute runoff volumes and associated pollutant loads in the proposed condition. As such, the assumed runoff coefficients are believed to be somewhat conservatively selected and reliable for the purpose of impact analysis.

Parcel-based BMP Infiltration Feasibility Screening

The types of parcel-based BMPs applied to commercial, multi-family, institutional, recreation and park land uses was determined based on infiltration feasibility constraints, as described in Section 2.1.3. The criteria used to categorize parcel-based treatment based on infiltration constraints are in agreement with the infiltration constraints listed in the *Ventura Technical Guidance Manual* (Ventura County, 2010), the *LA County LID Ordinance and Manual* (LACDPW, 2009), and the *LID BMP Design, Investigation and Reporting Requirements Administrative Manual* (LACDPW, 2011). Constraints were mapped as accurately as possible at the Tier 2 level of analysis. More detailed site investigation performed at later project phases may result in somewhat different distributions of parcel-based BMPs.

Parcel-based BMP Infiltration Rates

Infiltration rates beneath parcel-based BMP were assumed based on input from project geotechnical consultant (Seward, 2011) based on review of geologic information and proposed sources of fill material. While it is expected that infiltration rates may vary across the Project, the assumed values are believed to be representative of anticipated average conditions. Detailed designs will be supported by site-specific infiltration testing and will generally be based on the same design goals used to develop the parcel-based BMP designs simulated in this analysis.

Parcel-based BMP Model Representations

For the purpose of estimating the characteristic hydraulic performance (capture efficiencies and volume reductions) of parcel-based BMPs, detailed hydraulic representations were simulated to manage runoff from hypothetical spatially-averaged catchments. The spatially-averaged hypothetical approach provides representative and reliable estimates of hydraulic performance for two key reasons. First, the sizes of parcel-based BMPs scale linearly with tributary impervious area, there it is expected that the nearly identical capture efficiency and volume reduction (as a percent of total runoff volume) would be expected for catchments with a wide range of tributary area impervious fraction. Second, parcel-based BMP designs include significant “equalization storage” above their treatment layer, therefore the effect of catchment size (i.e., time of concentration) is not believed to be sensitive in the estimation of hydraulic performance. Therefore the use of a hypothetical, spatially-averaged catchment is appropriate to generate these inputs.

In order to size parcel-based BMPs for the purpose of analysis, BMP geometries were assumed based on the assumed underlying infiltration rate and the Project design goals for parcel-based BMPs. While the geometry assumed for this analysis is specific to a certain BMP design, the resulting performance parameters derived from this representation are reasonably representative of the hydraulic performance of a wide range of parcel-based BMPs provided that the Project design goals for parcel-based BMPs remain the same.

Drainage Area Routing and Nested BMP Representations

Each sub-regional facility drainage area includes areas treated by parcel-based BMPs and/or SFD HSCs. Because the exact location and detailed designs of these parcel-based BMPs and HSCs are not known at the Tier 2 level of analysis, it would be inappropriate to simulate detailed drainage area hydraulic routing to account for these nested BMPs. However, it would also be inappropriate to ignore the role of nested BMPs in the hydraulic performance of downstream region infiltration/ biofiltration facilities. The approach described in Section 2.1.7.3 balances these considerations to provide a reliable estimate of the hydraulic performance of sub-regional facilities that is consistent with the Project performance standards.

Sub-regional Facility Infiltration Rates and Model Representations

Infiltration rates in the locations of proposed sub-regional bioinfiltration/biofiltration facilities were estimated based on geologic information, soils data, and limited infiltration testing results available at the time of analysis. To account for uncertainty in these estimates, substantial factors of safety were applied. As such, it is believed that infiltration rates are somewhat conservatively selected for the purpose of this analysis and it is anticipated that higher design infiltration rates may be supported through site-specific analysis conducted at the time of the final hydrology report. Should detailed testing show infiltration rates are lower than assumed, additional design features such as dry wells and/or selectively graded fill material could be used to achieve at least the assumed design infiltration rate.

BMP Effluent Statistics

BMP effluent concentrations are based on studies contained in the most recent version of the International BMP Database. These studies are screened to remove data for undersized (i.e., inadequate design criteria) BMPs that are likely to have pollutant removal performance substantially less than the BMPs to be constructed for the Project. This screening is believed to improve the accuracy of BMP performance estimates; however, it is only intended to remove BMPs that are clearly unrepresentative in terms of sizing. The screening process is intended to include BMPs with adequate performance that may not be as well designed or maintained as the structural BMPs for the Project. It is anticipated that the BMPs for the Project will perform as well, if not better than, the projected performance based on the ASCE International BMP Database.

Assumption of No Correlation between Model Parameters

The water quality model randomly selects stormwater pollutant concentrations independent of the storm depth or antecedent dry period for each storm event modeled. The validity of the assumption of independence between variables is supported in Appendix B of the Landmark Village WQTR Appendix B. In general, no consistent level of correlation has been demonstrated between stormwater EMCs and rainfall depth or the antecedent dry period.

The assumption of independence of model parameters is believed to result in representative or somewhat conservative estimates post-developed runoff quality and loading, as well as somewhat conservative estimates of Project impacts. First, the empirical distribution of runoff EMCs implicitly includes events with a wide range of antecedent dry periods and event sizes. Therefore, the effects of antecedent dry period and storm depth are implicitly reflected in model estimates. Second, where weak correlations have been observed, concentrations tend to decrease with increasing storm depth. Because bypass from BMPs tends to occur more frequently in larger events and at the end of events, the assumption of no dependence would generally result in higher bypass concentrations, on average, than would be expected if these negative correlations were included. On these bases, random selection of stormwater pollutant concentrations, independent of storm depth and antecedent dry period, is believed to be the most reliable option for the modeling methodology at this level of analysis.

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Geosyntec Memorandum – Evaluation of Equivalency with Los Angeles County Low Impact Development Standards dated August 9, 2010

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Memorandum

Date:	09 August 2010
To:	Fred MacMurdo, Newhall Land
Copy to:	Mark Subbotin and Alex Herrell, Newhall Land
From:	Lisa Austin, Aaron Poresky, and Will Lewis, Geosyntec Consultants
Subject:	Landmark Village - Evaluation of Equivalency with Los Angeles County Low Impact Development Standards

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The purpose of this memorandum is to evaluate the performance of the approved stormwater BMP plan (BMP plan) for the Landmark Village Project (Project) (Vesting Tentative Tract Map 53108) in comparison to the requirements of the *Los Angeles County LID Standards Manual* (*LID Standards Manual*) (Los Angeles County, 2009), and to identify potential modifications to the BMP plan, if necessary, to achieve performance equivalent to the *LID Standards Manual*.

APPROVED BMP CONTROL PLAN

The stormwater treatment control BMPs planned for the Landmark Village Project are shown on *Landmark Village Tentative Tract Map 53108 Drainage Concept* (Psomas, 4/8/10). Treatment control BMPs consist of one extended detention (ED) basin, eleven bioretention areas, one vegetated swale, and two modular wetland units. The ED basin, identified as BMP-8, will provide treatment for 57 acres (21%) of the Project development area; the 11 bioretention areas will provide treatment for 204 acres (75%) of the Project development area; the vegetated swale will provide treatment for 3.5 acres (~1.5%) of the Project development area; and two modular wetlands will provide treatment for 2.6 acres (~1%) of the Project Development area.

These BMPs are designed per the requirements of the Newhall Ranch Sub-Regional Stormwater Mitigation Plan (NRSSMP) (Geosyntec, 2008). The applicable sizing criterion for the Project's ED basin and bioretention areas is the capture and treatment of 80 percent of the average annual runoff volume, with a drawdown time of 48 hours for the ED basin. This sizing criterion utilizes historical rainfall data with continuous simulation modeling and is consistent with volume-based sizing criterion #2 from the SUSMP Manual. The applicable sizing criterion for the Project's vegetated swale and modular wetlands uses a rainfall intensity based on SUSMP Appendix A,

with a minimum of 0.3 inches per hour, which meets or exceeds flow-based sizing criterion #1 from the SUSMP Manual.

LID EQUIVALENCY

Los Angeles County's *LID Standards Manual* (Los Angeles County, 2009) outlines stormwater runoff quantity and quality control development principles, technologies, and design standards for achieving the LID Standards of the Los Angeles County LID Ordinance (Los Angeles County, 2008). An analysis was performed to determine whether the LID and treatment control BMPs included in the Project would provide equivalent or greater volume reductions on an average annual basis to that which would be achieved by BMPs designed per the specific requirements of the *LID Standards Manual*. A detailed description of the analysis methodology for evaluating equivalency to the *LID Standards Manual* is provided in Attachment A.

To determine whether LID equivalency would be achieved, a two tiered analysis was conducted. The first tier of the analysis divided the Project area into three categories: (1) open space areas (which inherently have no mitigation requirements under the *LID Standards Manual*), (2) areas where infiltration is potentially feasible, and (3) areas were infiltration is likely infeasible based on infiltration feasibility criteria specified by the *LID Standards Manual*. Infiltration infeasibility screening was conducted by Seward Engineering Geology, Inc. (Seward, 2010) based on criteria contained in the *LID Standards Manual*. See Figure 1 for an illustration of the Tier One infiltration feasibility screening results. The analysis was conducted using conservative assumptions for the *LID Standards Manual* screening criteria so as to categorize more area as being potentially feasible for infiltration than would likely be found to be feasible during the detailed design phase. In this sense, this analysis should result in a conservative (more stringent) evaluation of LID equivalency.

The Tier Two analysis involved:

1) Calculation of the volume reduction that would be achieved by well designed BMPs per the specific requirements of the *LID Standards Manual*, with consideration for the infiltration feasibility screening conducted in the Tier One analysis. Volume reductions were calculated for hypothetical BMPs designed specifically to infiltrate (for those areas where infiltration is potentially feasible) and hypothetical vegetated BMPs designed to treat and release (for those areas where infiltration is potentially infeasible). The latter would likely achieve some volume reduction through incidental infiltration and evapotranspiration. *LID Standards Manual* sizing criteria were considered in these calculations.

2) Calculation of the volume reductions achieved by the Project's BMPs. Volume reductions were based on the type of proposed BMPs and the applicable NRSSMP sizing criteria for these BMPs. Volume reductions included incidental losses in BMPs designed to treat and release stormwater in vegetated BMPs and infiltration from BMPs designed specifically to infiltrate.

3) Comparison of these values.

4) Identifying revisions to the Project BMP plan to ensure LID equivalency, if necessary.

Table 1 provides the assumptions and calculations used to estimate the average annual volume reduction that would be achieved by implementing well designed BMPs per the specific requirements of the *LID Standards Manual* (i.e., the *LID Standards Manual* performance standard) to the Project. The *LID Standards Manual* performance standard is calculated to be an average annual volume reduction of approximately 49 acre-feet per year.

Table 1: LID Standards Manual Performance Standard Calculations

Feasibility Category	Open Space	Infiltration Feasible	Infiltration Infeasible	Total
Total Area, ac	39	70	184	**293**
Composite Imperviousness[1]	--	70%	61%	**--**
Average Annual Runoff Volume, ac-ft/yr	--	69	161	**230**
Average Annual Capture Efficiency of BMPs Designed per LID Standards Manual[2]	--	48%	48%	
Average Annual Volume Reduction of Captured Water for Vegetated BMPs[3]	--	100%	20%	
Performance Standard Average Annual Volume Reduction[4], ac-ft/yr	0	33	16	**49**

[1] Composite imperviousness based on distribution of land uses within each analysis area

[2] Capture efficiency estimated through continuous simulation modeling of 40 years of precipitation, runoff and routing for a hypothetical volume-based BMP sized per the *LID Standards Manual*

[3] Volume reduction assumption for vegetated treat and release BMPs per discussion in Attachment A.

[4] Volume Reduction = Total Runoff Volume * Capture Efficiency * Volume Reduction of Captured Water

The NRSSMP requires that BMPs be designed to capture and treat 80 percent of average annual runoff volume (Geosyntec, 2008). Of this captured volume, the fraction retained in vegetated BMPs and lost to evapotranspiration and/or incidental infiltration is assumed to be 20 percent on average. This is the same assumption used for hypothetical vegetated BMPs in establishing the *LID Standards Manual* performance standard. Additional volume reductions can be achieved by

designing BMPs to infiltrate 100 percent of captured water, where feasible. This would be accomplished through selective grading below these BMPs (where fill thicknesses allow) to enhance infiltration rates or by routing treated discharge from these BMPs to dry wells to be infiltrated below the compacted fill layer. Design enhancements to promote infiltration would be incorporated into the Project plans in the final hydrology design phase.

The Project would be able to demonstrate equivalency to the *LID Standards Manual* by conditioning the BMP plan to provide infiltration of runoff from approximately 13 acres of commercial land use and 3 acres of multi-family land use, or infiltration of runoff from any another combination of land uses providing equivalent average annual volume reduction. Calculations describing this equivalency scenario are provided in Table 2.

Table 2: Volume Reductions Achieved by Project BMPs

Volume Reduction Category	Vegetated Treat and Release BMPs Achieving Incidental Volume Reduction	BMPs Designed to Infiltrate Captured Water (Commercial and/or Multi-Family Land Uses)[5]	Total
Developed Land Uses Tributary to Project BMPs[1], ac	237	16	253
Composite Imperviousness of Developed Land Uses Tributary to Project BMPs[2]	62%	88%	
Total Runoff Volume from Developed Land Uses Tributary to Project BMPs , ac-ft/yr	211	19	
NRSSMP Capture Efficiency	80%	80%	
Volume Reduction of Captured Water Achieved by Project BMPs [3]	20%	100%	
Average Annual Volume Reduction Achieved[4], ac-ft/yr	**34**	**15**	**49**
LID Standards Manual Performance Standard Volume Reduction, ac-ft/yr (from Table 1)			**49**

[1] Includes developed land uses only. Some open space land uses may be tributary to Project BMPs, however these land uses do not have an *LID Standards Manual* mitigation requirement. Not accounting for runoff from open space land uses in volume reduction calculations results in a conservative estimate of volume reduction achieved in Project BMPs.
[2] Composite imperviousness based on distribution of land uses within each analysis area
[3] Volume reduction assumption for vegetated treat and release BMPs per discussion in Attachment A.
[4] Volume Reduction = Total Runoff Volume * Capture Efficiency * Volume Reduction of Captured Water
[5] Scenario includes infiltration of runoff from 13 acres of commercial land use and 3 acres of multi-family land use; other configurations may be evaluated as part of final hydrology calculations to achieve equivalent volume reduction.

Table 3 provides the distribution of commercial and multi-family land uses within areas considered potentially feasible for infiltration. Based on the areas where infiltration BMPs are potentially feasible, it will be practicable to modify the BMP plan in the final hydrology design

phase to provide the additional infiltration necessary to provide LID equivalency. Alternative configurations could be employed to achieve equivalent volume reductions, such as (1) infiltrating runoff from a lesser area of commercial land use and compensating by infiltrating runoff from a greater area of multi-family land use, and/or (2) enhancing the volume reduction achieved in the vegetated treat and release BMPs.

Table 3: Acreage of Commercial Land Use Potentially Feasible for Infiltration

Land Use	Surface Infiltration BMPs or Dry Wells Potentially Feasible, acre	Dry Wells Potentially Feasible, acre	Total, Infiltration Feasible, acre
Commercial/ Business Park	8.7	5.2	13.9
Multi-Family	5.0	14.4	19.4

Note: Infiltration BMPs are potentially feasible in portions of other land uses not shown in this table.

Based on this analysis, the approved Project treatment control BMPs, with modifications to incorporate infiltration BMPs or to enhance the infiltration capacity of the treat and release vegetated BMPs during final hydrology design would result in equivalent average annual volume reduction to that which would be achieved by BMPs designed per the requirements of the *LID Standards Manual*, thus would be deemed to be equivalent to the intent and requirements of the *LID Standards Manual*.

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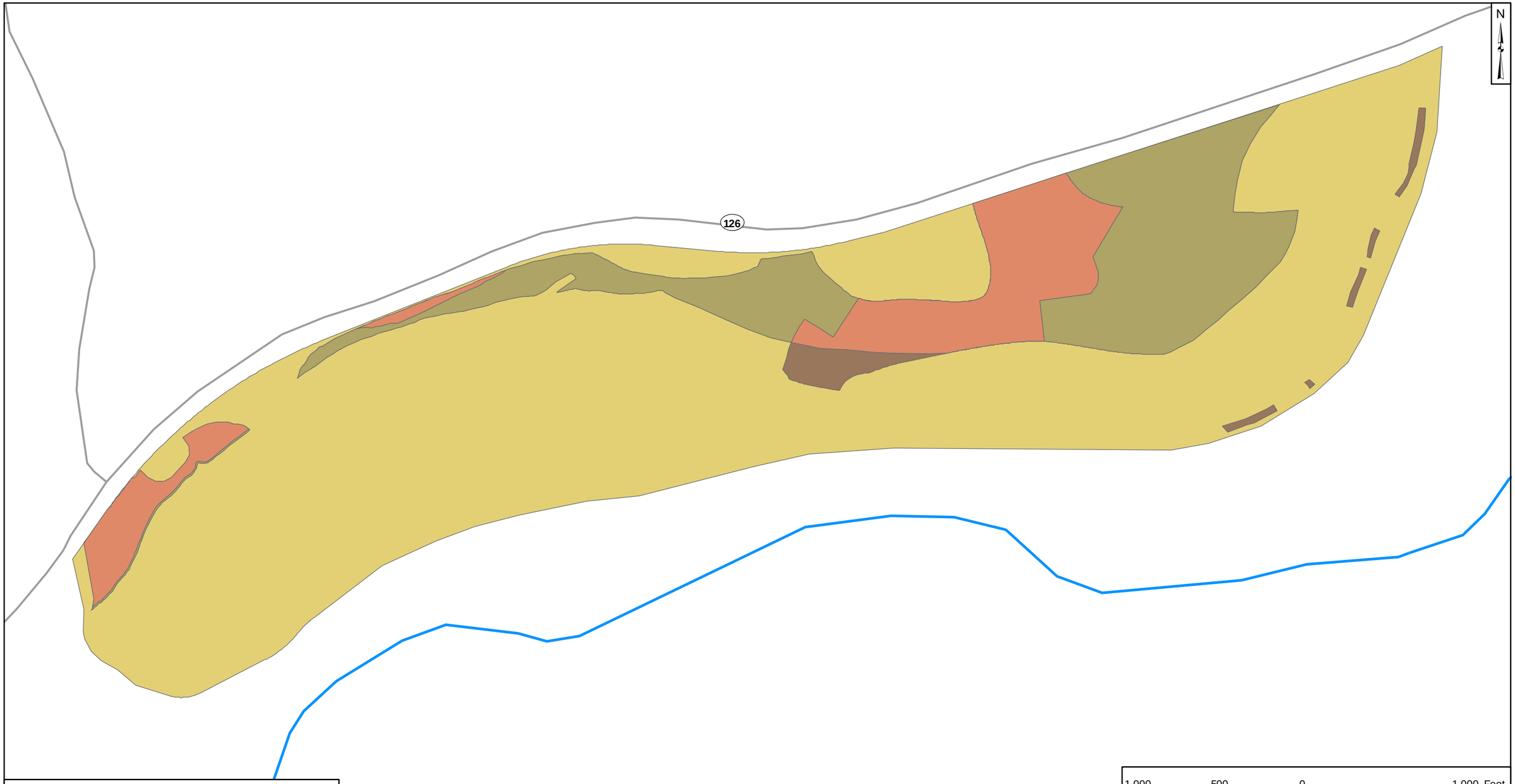

Legend

- Santa Clara River
- Major Road

Infiltration BMP Feasibility

- Dry Wells Potentially Feasible
- Dry Wells or Surface Infiltration Potentially Feasible
- Surface Infiltration Potentially Feasible
- Infiltration Likely Infeasible

1,000 500 0 1,000 Feet

Infiltration Feasibility and BMP Screening

Landmark Village LID Equivalency Analysis

Geosyntec consultants

Oakland | August 2010

Figure **1**

P:\GIS\Newhall\Projects\Landmark\Fig1_LID_Equiv_Analysis.mxd

Attachment A

LID Equivalency Analysis Methodology

ANALYSIS OF EQUIVALENCY OF NEWHALL RANCH SUB-REGIONAL STORMWATER MANAGEMENT PLAN (NRSSMP) PROVISIONS TO *LID STANDARDS MANUAL* REQUIREMENTS

An approach has been developed to compare the provisions of the *Newhall Ranch Sub-Regional Stormwater Management Plan* (NRSSMP) (Geosyntec, 2008) to the requirements of the Los Angeles County *LID Standards Manual*. In this approach, the performance of BMPs design per the NRSSMP performance standard is compared to the performance of BMPs sized per the *LID Standards Manual* requirements. The approach is implemented through a two tiered analysis. The first tier of analysis involves spatial data processing to divide the Project area into analysis regions based on the proposed developed condition and *LID Standards Manual* infeasibility criteria. The second tier involves calculating the volumetric performance of BMPs design per the *LID Standards Manual*, considering feasibility screening completed in Tier One, and calculating the volumetric performance of BMPs designed per the NRSSMP performance standard, and comparing these values.

Tier One Analysis Methodology

Tier One analysis utilizes spatial datasets and Geographic Information System (GIS) processing to divide the Project into analysis regions based on the *LID Standards Manual* infeasibility criteria listed below. A description of the steps in Tier One analysis follows.

LID Standards Manual Feasibility Criteria

LACDPW has developed a *LID Standards Manual* that outlines stormwater runoff quantity and quality control development principles, technologies, and design standards for achieving the LID Standards of Chapter 12.84. The *LID Standards Manual* requires that large scale residential and nonresidential development projects prioritize the selection of BMPs to treat stormwater pollutants, reduce stormwater runoff volume, and promote groundwater infiltration and stormwater reuse in an integrated approach to protecting water quality and managing water resources. The volumetric criterion associated with this requirement is the excess volume (ΔV), defined as the post-developed runoff volume minus the pre-developed runoff volume for the 85th percentile storm event (0.75 inches for Los Angeles County).

The Manual states that BMPs should be implemented in the following order of preference:

- BMPs that promote infiltration.
- BMPs that store and beneficially use stormwater runoff.
- BMPs that utilize stormwater runoff for other water conservation uses including, but not limited to, BMPs that incorporate vegetation to promote pollutant removal and runoff volume reduction and integrate multiple uses, and BMPs that percolate runoff through engineered soil and allow it to discharge downstream slowly.

If compliance with the above LID requirements is technically infeasible, in whole or in part, the project must incorporate design features demonstrating compliance with the LID requirements to the maximum extent practicable. The LID goals of increasing groundwater recharge, enhancing water quality, and preventing degradation to downstream natural drainage courses will be considered by DPW in the determination of infeasibility. The *LID Standards Manual* outlines site conditions where infiltration may not be possible:

- Locations where seasonal high groundwater is within 10 feet of the surface.
- Within 100 feet of a groundwater well used for drinking water.
- Brownfield development sites or other locations where pollutant mobilization is a documented concern.
- Locations with potential geotechnical hazards as outlined in a report prepared and stamped by a licensed geotechnical engineer.
- Locations with natural, undisturbed soil infiltration rates of less than 0.5 inches per hour that do not support infiltration-based BMPs.
- Locations where infiltration could cause adverse impacts to biological resources.
- Development projects in which the use of infiltration BMPs would conflict with local, State or Federal ordinances or building codes.
- Locations where infiltration would cause health and safety concerns.

The *LID Standards Manual* outlines where storage and reuse of the ΔV may not be possible:

- Projects that would not provide sufficient irrigation or (where permitted) domestic grey water demand for use of stored runoff due to limited landscaping or extensive use of low water use plant palettes in landscaped areas.
- Projects that are required to use reclaimed water for irrigation of landscaping.
- Development projects in which the storage and reuse of stormwater runoff would conflict with local, State or Federal ordinances or building codes.
- Locations where storage facilities would cause potential geotechnical hazards as outlined in a report prepared and stamped by a licensed geotechnical engineer.
- Locations where storage facilities would cause health and safety concerns.

Steps in Tier One

Establishing Areas with a ΔV>0

Proposed land use datasets are queried for areas that will undergo development. Areas that will remain as open space or will be impacted by development but restored to open space conditions will inherently have a ΔV of 0 and are not considered further in the equivalency analysis. Water quality treatment features are considered as open space for screening purposes.

Subdividing Areas with a ΔV>0 by Treatment Feasibility

Developed land uses are subdivided into the following feasibility categories:

1. None of the infiltration screening factors listed in the *LID Standards Manual* apply, thus infiltration is potentially feasible.

2. Infiltration is infeasible based on the criteria listed in the *LID Standards Manual*, but storage and reuse may be feasible.

3. Both infiltration and storage and reuse are infeasible based on the criteria listed in the *LID Standards Manual*.

A series of spatial analyses are performed to identify areas within the Project boundary that meet the numeric or narrative infeasibility criteria, as described below.

Establishing Areas Infeasible for Infiltration

Areas With Seasonally High Groundwater

Spatial datasets representing contours of depth to seasonally high groundwater[1] less than or equal to 10 feet provided by the Project's geotechnical consultants are intersected with developed areas within the Project boundary to yield the area where infiltration is infeasible due to seasonally high groundwater.

Areas Within 100 feet of a Drinking Water Supply Well

Infiltration infeasibility due to proximity to water supply wells in the Project area are considered if drinking water supply wells are present.

[1] The elevation to which the ground or surface water can be expected to rise during a normal wet season.

Brownfield Development Sites Where Pollutant Mobilization is a Concern

Infiltration feasibility due to the presence of brownfields does not apply to Newhall Land projects as there are no known contaminated soils or groundwater areas within the project boundaries.

Areas Where Natural Soil Infiltration Rates are Less Than 0.5 inches per hour:

The project's geotechnical consultant performs a review of geologic formations and available permeability testing data. Based on this review, the project area is divided into three categories:

1. Areas where permeability testing indicates that the native soils have a permeability rate of greater than 0.5 in/hr,
2. Areas that have not been tested where geologic conditions indicate that native soils may have a permeability of greater than 0.5 in/hr, and
3. Areas where permeability testing at limited locations and geologic conditions indicates that native undisturbed soils likely have permeability rate less than 0.5 in/hr.

The first two categories are assumed to be feasible for infiltration. The third category is assumed to be infeasible for infiltration. The third category is intersected with areas to be developed within the project boundary to yield the area where infiltration is infeasible due to an undisturbed soil infiltration rate likely to be less than 0.5 inches per hour.

Areas Where Geotechnical Hazards are Outlined by a Licensed Geotechnical Engineer

Spatial datasets representing areas where infiltration of stormwater could potentially result in geotechnical hazards are prepared by the project's geotechnical consultants. The project geotechnical consultants consider factors including, but not limited to, presence of landslides that will be left partially or entirely in place after remedial grading, depth of fill, and proximity to cut and fill slopes. Methods and findings are documented in reports and exhibits prepared by the project geotechnical consultants.

Where Infiltration May Cause Adverse Impacts to Biological Resources

Infiltrating the excess volume (ΔV) in some areas may lead to the creation of springs or seeps and/or unseasonal flows in ephemeral tributaries to the Santa Clara River. Spring/seeps and/or unseasonal flows may adversely impact ecosystems adapted to episodic precipitation and event-driven ephemeral flows. Infiltration infeasibility due to adverse impacts to biological resources may be considered if a sophisticated hydrogeologic analysis is available for the project that identifies the potential for the creation of springs or seeps in ephemeral channels within the project.

Areas Where the Use of Infiltration BMPS May Conflict with Building Codes

Current County guidance on setbacks for infiltration facilities includes:

- Property lines & Public Right of Way: 5 ft
- Any foundation: minimum of 15 ft or within a 1:1 plane drawn up from the bottom of foundation
- Face of any slope: minimum of 5 ft or H/2 (H is height of slope), unless otherwise recommended by a Soils Engineer and approved by Geotechnical and Materials Engineering Division.

It is anticipated that complete retention of the excess volume (ΔV) would not be feasible in some land uses due to these considerations. However, for this analysis, infiltration infeasibility due to potential conflict with building codes is not incorporated into spatial processing due to lack of detail in proposed development spatial data layers. It is expected that conflicts with building codes would be evaluated in the final hydrology study.

Areas Where Infiltration May Cause Health or Safety Concerns

At this time, no local, State, or Federal ordinances or statutes that may limit the feasibility of infiltration due to health and safety concerns have been identified. Therefore, infiltration feasibility due to health and safety is not considered in this analysis.

Merging and Subtracting to Establish Areas Where Infiltration is Infeasible and Feasible

All spatial datasets representing areas where infiltration is infeasible are merged in the order presented above and dissolved to represent a single surface representing areas with $\Delta V = 0$ (i.e., open space), areas where infiltration is potentially feasible, and areas where infiltration is potentially infeasible based on one or more screening criteria. Merging accounts for any overlapping areas and ensures that double counting does not occur when an area of infiltration feasibility or infeasibility within the developed areas of the project is established.

Establishing Areas with Storage and Reuse Infeasibility

Where infiltration is infeasible, the next priority is to store and use ΔV. As the Newhall Ranch projects are all required to use reclaimed water for irrigation of landscaping, storage and use of stormwater (rainwater harvesting) is not required per the feasibility criteria listed in the *LID Standards Manual*.

LID Where Infiltration and Retention and Storage Are Infeasible

Where both infiltration and harvesting are not feasible, ΔV must be treated in vegetated BMPs. The remaining water quality design volume may be treated in a non-vegetated BMP if necessary.

Tier Two Analysis Methodology

The Tier Two analysis sets a performance standard for the project based on the *LID Standards Manual* requirements and compares the performance of project BMPs designed to the NRSSMP standards to the *LID Standards Manual* performance standard. The *LID Standards Manual* performance standard is set based on the estimated long term performance of properly designed LID BMPs per the *LID Standards Manual*, considering the feasibility criteria described above.

LID Standards Manual Mitigation Volume Requirements Calculation

The *LID Standards Manual* prescribes the following six step method for computing the excess volume:

Step 1: Determine Hydrologic Parameters

A) Establish the area that will be disturbed and what areas will be left as open space.

B) Determine the length of flow path and calculate an average slope.

C) Identify soil type

Step 2: Establish a Design Storm

A) Identify the 85th percentile 24-hour runoff event.

Step 3: Calculate Undeveloped Runoff Volume for the Area to be Disturbed

A) Estimated using the LACDPW T_c Calculator based on area, proportion of impervious area, soil type, associated rainfall isohyet, and flow path length and slope as inputs.

Step 4: Calculate Developed Runoff Volume for the Area to be Disturbed:

A) Estimated using the LACDPW T_c Calculator based on area, proportion of impervious area, soil type, associated rainfall isohyet, and flow path length and slope as inputs.

Step 5: Calculate Excess Runoff Volume (ΔV)

A) Subtract undeveloped runoff volume from the developed runoff volume to establish ΔV.

Step 6: Determine Water Quality Treatment Volume or Flow Rate

A) Infiltrate, capture and reuse, or treat and release ΔV depending on feasibility criteria.

Alternative Approach for Establishing LID Mitigation Volume Requirements

The approach described above relies on values (i.e., length of flow path, slope, drainage patterns, etc.) that are not commonly available at a planning level. Therefore, a simplified approach utilizing runoff coefficients is employed for this stage of development planning as an alternative to the approach described above. Establishing runoff coefficients that would likely underestimate undeveloped runoff volume and overestimate developed runoff conditions would yield a ΔV greater than one established using the more site-specific LACDPW T_c calculator, thus providing a conservative (high) estimate of the *LID Standards Manual* performance standard.

A runoff coefficient of 0.1 was selected for the calculation of runoff from undeveloped areas (LACDPW, 2006). This runoff coefficient is the minimum runoff coefficient recommended for undeveloped areas in Chapter 6 of the LACDPW Hydrology Manual. LACDPW Hydrology Manual equation 6.3.2 for determining developed runoff coefficients (Equation 1) was simplified by setting the undeveloped runoff coefficient to 0.10. The resulting equation (Equation 2) was used to estimate runoff volumes in the existing and proposed conditions:

LACDPW Hydrology Manual: $C_d = (0.9 \times Imp) + (1 - Imp) \times C_u$ (Eqn. 1)

Simplified equation where C_u=0.10 $C_d = (0.8 \times Imp + 0.1)$ (Eqn. 2)

Where:

(C_d) = Developed runoff coefficient

(C_u) = Undeveloped runoff coefficient

Imp = Percent impervious

The existing condition of the project is assumed to be entirely undeveloped. While some development exists within the project area (e.g., roads and oil extraction areas), this assumption has the effect of increasing the *LID Standards Manual* performance standard and is therefore conservative. The resulting excess runoff volume for the project can be calculated as:

$\Delta V = V_{proposed} - V_{existing}$

$= d*A*[(0.8*Imp + 0.1)] - d*A*(0.1) = d*A*0.8*Imp$ (Eqn. 3)

Where,

d = design storm depth = 0.75 inches

A = tributary area (acres)

Imp = impervious fraction (ranges from 0 to 1)

This equation reduces to:

$$\Delta V \text{ (ac-ft)} = 0.05*A*imp \qquad \text{(Eqn. 4)}$$

Calculating Performance Associated with LID Standards Manual Mitigation Volume Requirements

To provide a basis for establishing equivalency, the performance of BMPs designed per the requirements of the *LID Standards Manual* is estimated using a three-part process:

1. Estimate the total average annual runoff volume from developed regions of the project,

2. Divide this volume between areas where infiltration is feasible and infiltration is not feasible,

3. Compute the average annual volume reductions that would be expected for BMPs designed to the standards of the *LID Standards Manual* and apply these reductions to the total runoff volumes computed in (2).

Land use acreages, imperviousness and infeasibility categories tabulated from the Tier One analysis are used to complete the first two steps. The simplified runoff coefficient equations described above are used with an approximate annual average rainfall depth of 18 inches per year[2] to calculate the average annual runoff volumes for developed areas of the project.

In order to estimate the performance of BMPs designed per the requirements of the *LID Standards Manual*, continuous simulation of a hypothetical catchment was conducted. First, a hypothetical BMP was sized to the *LID Standards Manual* requirements (Eqn. 4) for a hypothetical developed catchment. Then USEPA Stormwater Management Model (SWMM) was used to simulate the performance of this BMP over 40 years of historic hourly precipitation records[2]. This analysis assumed a 48-hour drawdown time of the stored volume, which is consistent with the *Los Angeles County Stormwater BMP Design and Maintenance Manual* (LACDPW, 2009). Table 1 reports the SWMM input parameters used for this simulation.

Table 1: SWMM Simulation Input Parameters

SWMM Parameters	Units	Values
Wet time step	seconds	600
Wet/dry time step	seconds	600
Dry time step	seconds	14,400
Precipitation	inches	733 (Patched Newhall Gage, COOP 046162, 10/1/1968-10/1/2008)[2]
Impervious Manning's n		0.012

[2] Full discussion of inputs and methodology are contained in NRSP Sub-Regional SWMP, Appendix B (Geosyntec, 2008).

SWMM Parameters	Units	Values
Hypothetical drainage area	acres	1
Shape		Rectangular, 250 ft flow path length
Impervious fraction modeled		100%
Slope	ft/ft	0.03
Evaporation	in / mo	60% of reference ET values for CIMIS Zone 14
Depression storage, impervious	inches	0.02, based on Table 5-14 in SWMM manual (James and James, 2000)
BMP Storage Volume	ac-ft	0.05 = 1 ac*0.6*100%imp/12 (Eqn. 4)
BMP Storage Volume	cu-ft	2,178
Drawdown Rate	cfs	0.0126 = 2,178 cu-ft/(48 hrs * 3600 sec/hr)

The resulting capture efficiency (i.e., the fraction of average annual runoff that is captured and not immediately bypassed by the BMP) was estimated to be approximately 48 percent based on this hypothetical scenario. The assumed impervious fraction of 100 percent is not important for this analysis because both runoff volume and modeled BMP volume have approximately linear dependency on impervious fraction. Sensitivity analyses show that conducting the same analysis with a lower impervious fraction would tend to yield lower capture efficiency and therefore set a lower *LID Standards Manual* volumetric performance standard.

Of the volume "captured," a portion is expected to be retained and a portion is expected to be released to the downstream conveyance system. For infiltration BMPs, captured water is expected to be fully retained up to the design storm event, therefore the total average annual reduction of runoff volume will be equal to the capture efficiency. In areas where infiltration is infeasible, vegetated treatment BMPs may still achieve incidental volume reductions through soil soaking and drying processes (i.e., evapotranspiration) and slower infiltration (unless facilities have an impermeable liner). An analysis of the International BMP Database (Strecker et al., 2004) found that detention basins and biofilters (swale and filter strips) achieved average volume reductions of 30 to 38 percent of captured volume, respectively. This analysis likely included studies of BMPs underlain by highly infiltrative soils. For project areas where infiltration is not feasible, it is likely that incidental volume reduction achieved by vegetated BMPs would be significantly less than that indicated by the Strecker et al. (2004) study. Therefore, for areas of the Project where infiltration is not feasible, it is assumed that 20 percent of the volume captured in vegetated BMPs is retained and lost to either evapotranspiration or incidental infiltration. The remaining 80 percent of captured volume is assumed to be treated and released. These values are multiplied by the average annual capture efficiency to yield the total reduction in average annual runoff volume.

In summary, the *LID Standards Manual* volumetric performance standard is computed as:

$$VR = RV \times \%Cap \times \%VR$$

Where: VR = average annual volume reduction achieved by BMPs designed per the *LID Standards Manual*

RV = the average annual runoff volume without BMPs for developed areas of the Project, divided between areas where infiltration is feasible and areas where infiltration is infeasible

%Cap = long term average capture efficiency = 48% (from SWMM modeling results for hypothetical catchment presented above)

%VR = long term average volume reduction of captured water = 100% (where infiltration is feasible) and 20% (where infiltration is infeasible, derived from Strecker, et al., 2004)

Volume reductions are computed independently for areas where infiltration is likely feasible and where infiltration is likely infeasible. These values are summed to yield the total volumetric performance standard for the project:

$$VR_{PROJECT} = VR_{INF\ FEASIBLE} + VR_{INF\ INFEASIBLE}$$

$VR_{PROJECT}$ is expressed in units of acre-feet per year.

Calculating Performance of Project BMPs

The NRSSMP requires that BMPs be designed to capture and treat 80 percent of average annual runoff volume. Of this captured volume, the fraction retained in BMPs and lost to evapotranspiration and/or incidental infiltration is assumed to be 20 percent on average. This is the same assumption used in establishing the *LID Standards Manual* performance standard (see previous section). Where site-specific information and BMP designs are available, the volume reduction achieved by project BMPs is calculated by performing continuous simulations in SWMM that account for infiltration and evapotranspiration.

Comparing Performance of Project BMPs to LID Standards Manual Performance Standard

The Tier Two methodology allows for the direct comparison of the volumetric performance of project treatment control BMPs and the *LID Standards Manual* performance standard. In instances where the performance of project BMPs meets or exceeds the *LID Standards Manual* performance standard, the project is deemed equivalent to the intent and requirements of the *LID Standards Manual*. Where project BMPs achieve a portion of the *LID Standards Manual* performance standard, the project would need to be conditioned to achieve the remaining portion of the *LID Standards Manual* performance standard, as feasible, through additional BMPs or enhancements to proposed BMPs.

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The Sanitation Districts' memorandum, along with the SCOPE letter and transcript of the oral testimony

SANITATION DISTRICTS OF LOS ANGELES COUNTY

STEPHEN R. MAGUIN
Chief Engineer and General Manager

Memorandum

Date: March 8, 2011

To: Supervisor Michael D. Antonovich, Mayor
Supervisor Gloria Molina
Supervisor Mark Ridley-Thomas
Supervisor Zev Yaroslavsky
Supervisor Don Knabe

From: Stephen R. Maguin SR Maguin
Chief Engineer and General Manager

Subject: **Response to SCOPE Letter and Testimony to the Board of Supervisors Regarding Formation of the Newhall Ranch Sanitation District (January 18, 2011 Board Agenda Item 25)**

On January 18, 2011, the Board of Supervisors (Board) approved a motion by Supervisor Antonovich directing the Sanitation Districts (Districts) to prepare a memorandum that responds to the issues raised by the testimony of Ms. Lynne Plambeck and Ms. Cam Noltemeyer of the Santa Clarita Organization for Planning and the Environment (SCOPE), and the letter from Ms. Plambeck dated January 13, 2011 (both documents are attached with issues numbered).

The memorandum presents background information on the proposed Newhall Ranch Development and the prior actions by the County and the Districts that provide context for the issues raised. SCOPE's issues are shown in bold followed by the Districts' response.

I. Background

On March 23, 1999, and, again, on May 27, 2003, the Board certified the environmental documents (collectively, Newhall Ranch EIR) for the Newhall Ranch Specific Plan and Newhall Ranch Water Reclamation Plant (NRWRP). The certified Newhall Ranch EIR evaluated the NRWRP at a project level of detail, and the Board approved the NRWRP under Conditional Use Permit No. 94-087-(5). The NRWRP is to provide treatment of the wastewater generated within the Specific Plan area as well as produce recycled water for the Specific Plan area.

The environmental analysis of the NRWRP is found in Section 5.0 of the Newhall Ranch Revised Draft EIR (March 8, 1999) and Section 3.0 of the Newhall Ranch Revised Additional Analysis, Volume VIII (May 2003). Section 3.0 assessed and updated various NRWRP alternatives including the approved NRWRP site.

The Newhall Ranch EIR contained a mitigation measure (Mitigation Measure 5.0-52), also reflected in the adopted Mitigation Monitoring Plan, requiring formation of a county sanitation district for the Specific Plan area. To fulfill mitigation requirements and establish a logical plan for development of the new district and its infrastructure, the Newhall Land and Farming Company (NLFC) and Sanitation Districts Nos. 26 and 32, later consolidated as Santa Clarita Valley Sanitation District (SCVSD), entered into an agreement (Interconnection Agreement) dated January 9, 2002.

On December 13, 2005, the Board adopted the resolution of intent to form the county sanitation district to be known as the Newhall Ranch County Sanitation District (NRSD). The Board also approved an Addendum to the Newhall Ranch EIR and Additional Analysis, which evaluated the environmental effects of NRSD formation. The Addendum determined that formation of the NRSD would not result in new or substantially more severe environmental impacts than those discussed in the prior Newhall Ranch environmental documents.

Thereafter, the County initiated proceedings for the formation of the NRSD, pursuant to the Cortese-Knox-Hertzberg Local Government Reorganization Act of 2000. On June 14, 2006, the Local Agency Formation Commission (LAFCO) for Los Angeles County adopted a resolution approving formation of the NRSD. On July 27, 2006, LAFCO issued a Certificate of Completion for formation of the NRSD.

On January 18, 2011, the Board considered a resolution confirming formation of the NRSD within the scope of the previously certified Newhall Ranch EIR and Addendum. At the January 18, 2011 Board meeting, representatives from SCOPE expressed their concerns by oral testimony and a letter.

II. Districts' Responses to SCOPE's Issues

1. "**Without the construction of the Sanitation plant as required by the Newhall Ranch Specific Plan, the public will bear the burden of the expensive clean up of chlorides required to comply with the Clean Water Act. This will entail a sharp increase in sewer fees to the general public."**

   Discharge of Newhall Ranch wastewater to the Valencia Water Reclamation Plant (VWRP) would be temporary until construction of the Newhall Ranch Water Reclamation Plant (NRWRP). The Newhall Ranch wastewater would neither add nor alleviate the SCVSD's financial burden to comply with the Upper Santa Clara River Chloride Total Maximum Daily load (Chloride TMDL).

   The Interconnection Agreement sets conditions under which the first 6,000 homes in Newhall Ranch may temporarily discharge wastewater to the VWRP. The conditions include payment of the standard SCVSD connection fee (fair share of the cost of the existing infrastructure) and transfer of title of the 22-acre NRWRP site to the NRSD. Newhall Ranch residents also would pay the SCVSD an annual service charge to recover the full cost of treating their wastewater at the VWRP. Temporary treatment of wastewater at the VWRP would not eliminate the need for the developer to construct the NRWRP. Prior to building more than 6,000 homes, the developer must construct the NRWRP.

   When operating at flows equal to or below the permitted plant capacity, compliance with the Chloride TMDL will depend on the chloride concentration in the treatment plant effluent. This concentration results from two primary sources: chloride concentration of the local water supply, and increased chloride concentration due to use of the water by the community. Local groundwater is the planned potable water source for the Specific Plan's Landmark and Mission Villages, the two developments whose wastewater might be temporarily treated at the VWRP under the Interconnection Agreement. The groundwater chloride levels for those communities are similar to that of the groundwater used by existing Santa Clarita Valley communities. Thus, no difference in chloride concentration is expected due to the water supply.

   Like Santa Clarita, Newhall Ranch will be a mixture of residential, commercial and industrial land uses. Use of automatic water softeners (AWS) was a significant chloride source for SCVSD wastewater prior to the 2008 ban on AWS. Per Specific Plan mitigation measure 5.0-52(b), the Newhall Ranch developer must request that NRSD ban AWS in Newhall Ranch. Districts' staff will also recommend that NRSD enact an AWS ban similar to the ban in the SCVSD. Consequently, the two communities are expected to produce similar increases in chloride concentrations due to use and similar overall wastewater chloride concentrations. Since final compliance will be determined by concentration, the addition of Newhall Ranch wastewater to the VWRP would neither add to nor alleviate the SCVSD's financial burden to comply with the Chloride TMDL.

2. **"…In addition, the agreement between the Santa Clarita Valley Sanitation District of Los Angeles County (SCV) and Newhall Land and Farming allows up to 6,000 capacity units to be treated at existing SCV wastewater treatment facilities as needed during construction of the Newhall Ranch Water Reclamation Plant. SCV has sufficient capacity to accommodate the use of its facilities." This statement cannot be made because the County is currently in the middle of analyzing the impacts for the first tract maps of Newhall Ranch. No certified EIR exists on either the Landmark tract or the Mission Village**

tract, which comprise approximately 6,000 units. Further, there is not even a Development Monitoring System analysis for sewer capacity included in the Mission Village EIR as required by the Court Decision in 2003."

Certification of an EIR is not required to estimate future flows and determine whether there is available capacity at existing treatment facilities. The 2003 Court Ruling by Judge Randall (Case Number S-1500-CV-239324, RDR) does not specify any requirements regarding a Development Monitoring System (DMS) analysis.

Wastewater flow projections for the two Newhall Ranch communities have been reviewed by the Districts. Estimates are 0.3 million gallons per day (mgd) for Landmark Village and 1.0 mgd for Mission Village (collectively 1.3 mgd). The Interconnection Agreement allows for temporary treatment at VWRP for up to 6,000 homes (about 1.6 mgd). The VWRP treated approximately 15 mgd in 2010 and currently has a capacity of 21.6 mgd (yielding 6.6 mgd of surplus capacity). Thus, the VWRP has sufficient capacity to accommodate the temporary use of its facilities as stated in the staff report for the January 18, 2011 Board agenda item. CEQA for the VWRP was addressed by the certified 2015 Santa Clarita Valley Joint Sewerage System Facilities Plan and EIR, which examined the environmental impacts of treating 27.6 mgd of wastewater at the VWRP.

The Newhall Ranch EIRs, certified by the Board in 1999 and 2003, evaluated the environmental impacts related to development of the Specific Plan, including construction of the NRWRP and the new sewage facilities to serve the Specific Plan area. At the project level, the County is in the process of completing further CEQA analysis for both Landmark Village and Mission Village. The CEQA compliance for Landmark Village is contained in the Landmark Village Draft EIR (November 2006), Final EIR (November 2007), and Recirculated Draft EIR (January 2010). CEQA compliance for Mission Village is contained in the Mission Village Draft EIR (October 2010). The EIRs contain a County DMS analysis and evaluate each project's wastewater conveyance/disposal effects including temporary wastewater treatment at the VWRP.

3. "**If the Sanitation Plant is not built in accordance with the mitigation requirements of the Newhall Ranch Specific Plan, the Plan cannot meet its requirements to provide non-potable water or to finance its own infrastructure expansion costs."**

   Temporary use of the VWRP for treatment of Newhall Ranch wastewater does not eliminate the Specific Plan requirement for the developer to construct the NRWRP and finance the new sewerage system. The temporary use of the VWRP addresses practical engineering considerations such as the need to build-up an adequate and steady flow of wastewater before start-up of the NRWRP. Whether Newhall Ranch wastewater is treated at the NRWRP or VWRP, the treated wastewater will be suitable for reuse and offsetting Newhall Ranch water demands.

4. "**Further, the Sanitation discharge permit granted by the Regional Water Quality Board required reverse osmosis treatment for the effluent from this plant. By attempting to evade this requirement, Newhall will put the added burden of removing salts from the Newhall Ranch effluent on the backs of the public."**

   Temporary use of the VWRP for treatment of Landmark Village and Mission Village wastewater does not eliminate the requirement for the developer to construct the NRWRP or finance the new sewerage system within the Specific Plan area. The developer must construct the NRWRP per the Specific Plan and must have it operating properly before the next phase after Landmark Village and Mission Village. As noted in the Item 1 response, temporary treatment of Landmark Village and Mission Village wastewater at the VWRP would neither add to nor alleviate the SCVSD's financial burden to comply with the Chloride TMDL.

   Temporarily treating wastewater from the first 6,000 Newhall Ranch homes at the VWRP is a practical engineering decision based on the need to build up an adequate, steady flow of wastewater before starting up

the NRWRP, especially the reverse osmosis units. Such an approach would match the slower pace of the development but would not eliminate the Specific Plan requirement for construction of the NRWRP.

5. "**The Santa Clarita Sanitation District's failure to meet the Clean Water TMDL standard for chloride of 100 mg/l in the Santa Clara River is a result in part due to the sharp and continuing increase in the use of imported State Water Project (SWP) water as seen by the chart below, (also supplied by the Sanitation Districts). This problem is aggravated by high levels of chlorides in the wells proposed to be used for these tracts, according to information found in both the Landmark and Mission Village DEIRs as indicated in the chart below. Therefore, if Newhall uses the Valencia treatment plant rather than building their own Sanitation Plant as required by the Specific Plan, the chloride levels in the effluent of that treatment plant will be substantially increased. Without the immediate construction of the Newhall Ranch Water Reclamation Plant, approved as an RO (reverse osmosis salt removal system) facility, the high chlorides in the wells proposed to be used by this project in the chart below and the additional imported Nickels water will add to this load."**

   Imported water did not cause the chloride standard to be exceeded. Effluent from the VWRP has exceeded 100 mgd/l since the 1970s despite the fact that imported water was not delivered to Santa Clarita Valley until the 1980s. Nonetheless, as noted in the Item 1 response, the chloride concentrations of Newhall Ranch and SCVSD wastewater are expected to be similar. Thus, temporary treatment of Newhall Ranch wastewater at the VWRP would not change the SCVSD's ability to comply with the Chloride TMDL.

   SCOPE implies that use of Nickel water[1] would contribute to increase the chloride load at the VWRP. While the Landmark Village and Mission Village projects are part of the potable water system for the entire Specific Plan, the projects do not rely on Nickel water to satisfy their potable water demands. As reported in the Newhall Ranch Revised Additional Analysis, Section 2.5, Water Resources (Volume VIII, May 2003), the Nickel water would only be needed in years when the Newhall Ranch agricultural water has been used, which is estimated to occur after approximately the 21st year of project construction. Therefore, the comment regarding use of Nickel water is not appropriate at this time.

6. **"How does a side agreement between the developer and the Sanitation Districts fit into the planning oversight purview of the Board of Supervisors? How can the Planning Department substantiate that sewer service complies with the County Development Monitoring System or is consistent with the general plan or specific plans if developers make side agreements with the Sanitation Districts?"**

   Formation of a new sanitation district was identified in the Specific Plan EIR as a mitigation measure. The Interconnection Agreement was developed to fulfill this Specific Plan requirement and establish a logical plan for the development and administration of the new district and its infrastructure. This agreement ensures that the developer provides the necessary land and infrastructure. The Interconnection Agreement was considered and approved by the Sanitation Districts Nos. 26 and 32 Boards at their January 9, 2002 meetings, which were open to the public. Further, this agreement was referenced in previous County and LAFCO resolutions supporting formation of the new sanitation district.

   As noted in the Item 2 response, the EIRs for both Landmark Village and Mission Village contain County DMS analysis. Moreover, the Newhall Ranch developer is required to build a new sewerage system to serve Newhall Ranch developments and, thus, the Specific Plan does not rely upon existing County sewerage facilities. The Districts and County have coordinated their efforts with regard to establishment of the new sanitation district and its sewerage conveyance system. This coordination enables the County to verify that the development is consistent with the County's General Plan and Specific Plan requirements.

[1] Nickel water refers to a source of potable water owned by NLFC that can be delivered to the Newhall Ranch development to supplement existing sources of potable water.

7. "The agreement between the developer of the Newhall Ranch Project and the Sanitation District violates the conditions of the Newhall Ranch Specific Plan and puts the Santa Clarita Valley in jeopardy of continued non-compliance with the Clean Water Act Chloride TMDL. We therefore strongly object to this agreement and ask that the Board of Supervisors take action to rectify this issue."

The Interconnection Agreement is not in conflict with the Specific Plan and does not impact the SCVSD's ability to comply with the Chloride TMDL. As noted in the Item 1 and 4 responses, temporary treatment of Landmark Village and Mission Village wastewater at the VWRP would not eliminate the need for the developer to construct the NRWRP and to finance the new sewerage system, nor would it impact compliance with the Chloride TMDL. As presented in the Item 2 response, the VWRP has available capacity for temporary treatment of Landmark Village and Mission Village wastewater. Thus, no negative impact to the SCVSD's sewerage system is expected, and this approach does not conflict with the Specific Plan's requirement for construction of the NRWRP.

8. "The public should not have to pay the costs of bringing the chloride level into compliance with an increase to their sewer fees."

By law, the users of the SCVSD's wastewater system must pay for Chloride TMDL compliance. As noted in the Item 1 and 4 responses, temporary treatment of Newhall Ranch wastewater would neither add to nor alleviate the SCVSD's financial burden to comply with the Chloride TMDL.

9. "…but for the statement within the resolution that says that "The first 6,000 units of Newhall Ranch will be put through the Valencia Treatment Plant." That's not consistent with the Newhall Ranch that was passed for the formation of this, the Newhall Ranch sanitation plant."

The temporary treatment of Landmark Village and Mission Village wastewater at VWRP does not conflict with Specific Plan's requirements as described in the Item 4 and 7 responses.

10. "And we ask that that be struck from the staff report because it seems to be a backdoor way of getting those approved when there's no E.I.R. on that and it's not consistent with the Specific Plan."

As noted in the Item 4 and 7 responses, temporary treatment of Landmark Village and Mission Village wastewater at the VWRP is not in conflict with the Specific Plan. Prior CEQA compliance was not required because temporary treatment at the VWRP was not proposed until the release of the Draft EIRs for both Landmark Village and Mission Village. Draft EIRs for both projects, including the Landmark Village Recirculated Draft EIR, have been the subject of extensive public review and comment as part of the County's environmental review process.

As stated in the Item 2 response, the environmental implications of the build-out of the VWRP to its capacity were assessed in the SCVSD's certified EIR for the 2015 Santa Clarita Valley Joint Sewerage System Facilities Plan. The Newhall Ranch EIR, evaluated the environmental impacts related to development of the Specific Plan, including construction of the NRWRP to a project level and the new sewerage facilities at a programmatic level to serve the Specific Plan. The County is in the process of completing further CEQA compliance at a project level for both Landmark Village and Mission Village.

11. "The addendum itself that … was passed … for the formation on the Sanitation District specifically says that the wastewater treatment plant will be built in stages as the specific plan area is developed and will ultimately be sized to treat up to 6.8 million gallons. So it, too, is not consistent with what is being said in the Staff Report. So we wonder how the Sanitation District would have made an agreement like that that's in violation of your environmental documents and the Specific Plan."

There is no inconsistency between the Staff Report and the Specific Plan. The fact that the Staff Report only addressed the temporary treatment of Newhall Ranch wastewater at the VWRP does not eliminate the Specific

Plan requirement for the developer to build the NRWRP and other sewerage infrastructure to serve the Specific Plan. For more information regarding consistency with the Specific Plan, see the Item 6 response. Regarding claims of violating CEQA, please see the Item 10 response.

12. **"Now we appear before you, and Newhall Land is claiming that they have this agreement with the Sanitation -- actually Santa Clarita Valley Sanitation District of Los Angeles County to allow these 6,000 units to be treated in our existing Santa Clarita wastewater facilities. Those facilities are not reverse osmosis plants. And if this is allowed, it will only create additional problem as far as the chlorides for our community. The reverse osmosis plant that is required with this Newhall Treatment Plant that will take care of chlorides. So definitely, they shouldn't be allowed to use any other treatment plant."**

    Discharge of Landmark Village and Mission Village wastewater to the VWRP will be temporary until construction and startup of the NRWRP. The Landmark Village and Mission Village wastewater would neither add to nor alleviate the SCVSD's financial burden to comply with the Chloride TMDL. For further explanation, see the Item 1 and 4 responses.

13. **"And it's a very, very expensive issue for our community. And we were promised that we would not be funding anything for the Newhall Ranch."**

    Temporary treatment of Landmark Village and Mission Village wastewater at the VWRP would neither add to nor alleviate the SCVSD's financial burden to comply with the Chloride TMDL as explained in the Item 1 and 4 responses.

14. **"And if that's what they're going to do, they have to have additional environmental analysis on it."**

    As noted in the Item 10 response, the EIRs for both Landmark Village and Mission Village evaluate wastewater disposal options including temporary treatment of Landmark Village and Mission Village wastewater at the VWRP. There will be no temporary treatment at the VWRP, unless and until the Board has considered and certified the project EIRs in accordance with CEQA.

Should you have any questions or concerns, please contact me, or Mr. Thomas J. LeBrun at (562) 908-4288, extension 2751 or via email at tlebrun@lacsd.org.

cc: Board of Directors – Santa Clarita Valley Sanitation District
Department of Public Works
Regional Planning Commission

SRM:TJL:ddg

WATER RECLAMATION
SOLID WASTE MANAGEMENT

COUNTY SANITATION DISTRICTS
OF LOS ANGELES COUNTY

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www.lacsd.org

STEPHEN R. MAGUIN
Chief Engineer and General Manager

March 8, 2011

Board of Directors
Santa Clarita Valley Sanitation District
of Los Angeles County

Directors:

Memorandum to Board of Supervisors – Newhall Ranch Sanitation District

Enclosed is the memorandum requested by the Board of Supervisors regarding formation of the Newhall Ranch Sanitation District and responses to the issues raised by the Santa Clarita Organization for Planning and the Environment.

Please contact me should you have any questions.

Very truly yours,

Stephen R. Maguin

Stephen R. Maguin

SRM:TJL:ddg

Enclosure

DOC # 1800048

Recycled Paper

SCOPE
Santa Clarita Organization for Planning and the Environment
TO PROMOTE, PROTECT AND PRESERVE THE ENVIRONMENT, ECOLOGY AND QUALITY OF LIFE IN THE SANTA CLARITA VALLEY
POST OFFICE BOX 1182, SANTA CLARITA, CA 91386

1-13-11

Attn: Executive Office
LA County Board of Supervisors
500 W. Temple St.
Los Angeles, CA 90012

Re: Agenda Item # 25 – Inconsistency with Newhall Ranch Specific Plan
Please copy to all Supervisors

Dear Sirs:

It has come to our attention that, while the staff report for this agenda item correctly states the timeline of the formation of the Newhall Ranch Sanitation District, it also includes erroneous information and brings to light an agreement made between the Sanitation Districts and Newhall Land and Farming that is inconsistent with the Newhall Ranch Specific Plan. Further, it misinforms the Board as to the financial impacts of such an agreement.

We ask that the Supervisors, and particularly Mr. Antonovich, as our representative on the Board of Sanitation District 26 and 32, immediately investigate and set aside this agreement. We request that Board of Supervisors, as ultimate oversight authority for the approval and conformity of the Newhall Ranch Specific Plan, object to this agreement between the Newhall Land Co. and the Sanitation Districts. We request that the Board delay approval of this agenda item until this investigation is completed and the staff report is corrected.

We particularly object to these two sections of the staff report:

1. **"FISCAL IMPACT/FINANCING**

   It is anticipated that the operation and maintenance of the District and its facilities would be funded through the imposition of service charges, which would be collected on the tax roll, and construction of the facilities would be financed by the developer for the Newhall Ranch project."

1 **Without the construction of the Sanitation plant as required by the Newhall Ranch Specific Plan, the public will bear the burden of the expensive clean up of chlorides required to comply with the Clean Water Act. This will entail a sharp increase in sewer fees to the general public.**

2. **"IMPACT ON CURRENT SERVICES (OR PROJECTS)**

2 This project will not have an adverse impact on current sewage services because the District will build facilities to serve all new developments within the Newhall Ranch Specific Plan area. In addition, the agreement between the Santa Clarita Valley Sanitation District of Los Angeles County (SCV) and Newhall Land and

Farming allows up to 6,000 capacity units to be treated at existing SCV wastewater treatment facilities as needed during construction of the Newhall Ranch Water Reclamation Plant. SCV has sufficient capacity to accommodate the use of its facilities."

2 **This statement cannot be made because the County is currently in the middle of analyzing the impacts for the first tract maps of Newhall Ranch. No certified EIR exists on either the Landmark tract or the Mission Village tract, which comprise approximately 6000 units. Further, there is not even a Development Monitoring System analysis for sewer capacity included in the Mission Village EIR as required by the Court Decision in 2003.**

Background

The Mitigation Monitoring Plan of the Newhall Ranch Specific Plan states that:
SP 4.11-1 The proposed Specific Plan **SHALL**[1] implement a water reclamation plant in order to reduce to specific plan's demand for imported potable water, The Specific Plan **SHALL** install a distribution system to deliver non-potable reclaimed water to irrigate land uses suitable to accept reclaimed water, pursuant to the Los Angeles County Department of Health Standards. **Mitigation 4.11-8 requires Newhall to pay for the cost of water expansion** by paying for connection fees and **Mitigation 4.12-7** ensured the public would not have to pay for the development of Newhall Ranch by requiring that future tracts would have to be annexed into a sewer district.
SP 4.12-2 A 5.8 to 6.9 mgd water reclamation plant **SHALL** be constructed on the Specific Plan site, pursuant to County, State, and Federal design standards, to serve the Newhall Ranch Specific Plan.
SP 4-12-3 A Conceptual Backbone Sewer Plan **SHALL** be implemented pursuant to County, state and federal design standards.

Please note: The mitigation monitoring system does NOT say "may", it says, "**SHALL**".

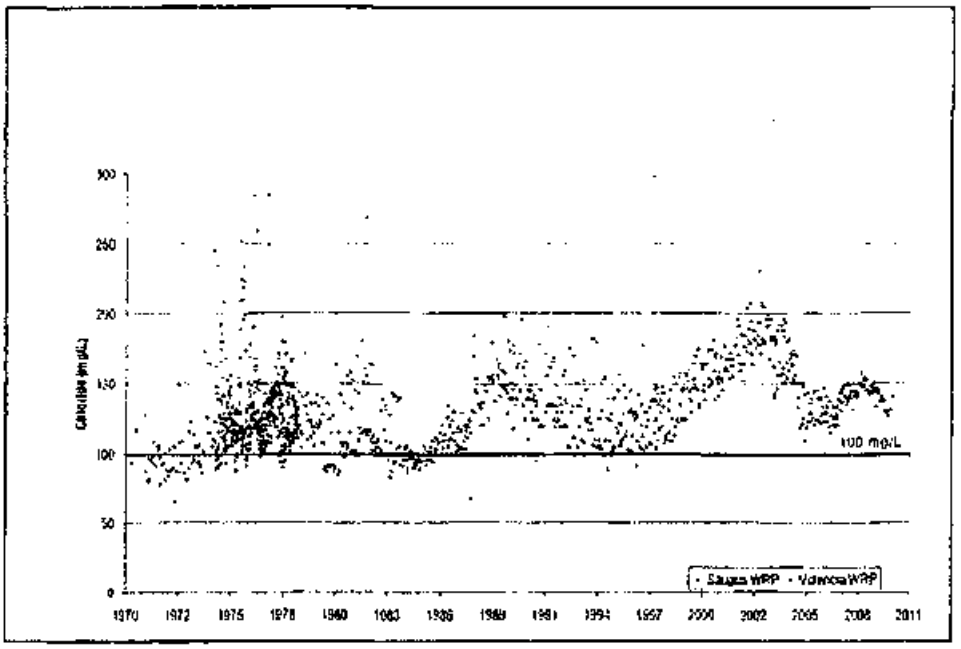
3 If the Sanitation Plant is not built in accordance with the mitigation requirements of the Newhall Ranch Specific Plan, the Plan cannot meet its requirements to provide non-potable water or to finance its own infrastructure expansion costs.

4 **Further, the Sanitation discharge permit granted by the Regional Water Quality Board required reverse osmosis treatment for the effluent from this plant. By attempting to evade this requirement, Newhall will put the added burden of removing salts from the Newhall Ranch effluent on the backs of the public.**

The CHLORIDE issue

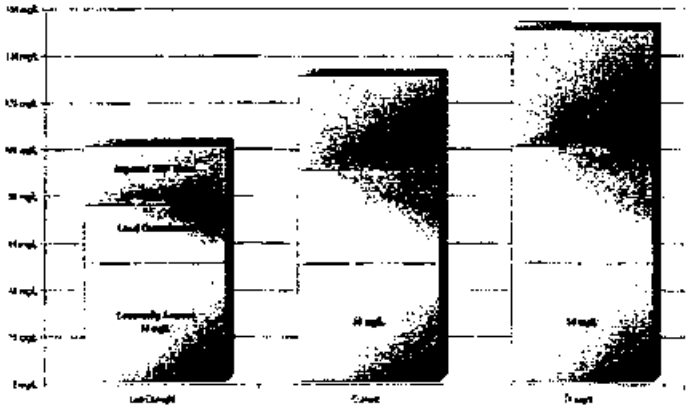
Currently the Sanitation Districts 26 and 32 in the Santa Clarita Valley do not comply with the Clean Water Act Act Total Maximum Daily Load (TMDL) effluent standard of 100 ugl for Cholride as indicated by the chart below supplied at a recent Sanitation District public hearing:

[1] Emphasis added to all "shalls" in this section



5 The Santa Clarita Sanitation Districts' failure to meet the Clean Water TMDL standard for chloride of 100mg/l in the Santa Clara River is a result in part due to the sharp and continuing increase in the use of imported State Water Project (SWP) water as seen by the chart below, (also supplied by the Sanitation Districts).

Chloride Sources During
Drought & Non-Drought Conditions



5 This problem is aggravated by high levels of chlorides in the well proposed to be used for these tracts, according to information found in both the Landmark and Mission Village DEIRs as indicated in the chart below. Therefore, if Newhall uses the Valencia treatment plant rather than building their own Sanitation Plant as required by the Specific Plan, the chloride levels in the effluent of that treatment plant will be substantially increased. Without the immediate construction of the Newhall Ranch Water Reclamation Plant, approved as an RO (reverse

osmosis salt removal system) facility, the high chlorides in the wells proposed to be used by this project in the chart below and the additional imported Nickels water will add to this load.

Water Quality Constituents of Concern
Secondary Standards:
(from Mission Village DEIR Appendix F4.8)

Parameter	MCL	DLR	Units	E-14	E-15	E-16	E-17
Chloride	250-500-600	NA	mg/L	75	68	69	74
	pH 6.5 - 8.5	NA	units	7.5	7.7	7.3	7.4
Specific Conductance (E.C.)	900-1600-2,200	NA	umho/cm	1240	1290	1390	1360
Sulfate	250-500-600	0.5	mg/L	340	330	340	340
Total Dissolved Solids (TDS)	500-1000-1500	NA	mg/L	900	890	950	960

Conclusion and Questions

How does a side agreement between the developer and the Sanitation Districts fit into the planning oversight purview of the Board of Supervisors? How can the Planning Department substantiate that sewer service complies with the County Development Monitoring System or is consistent with the general plan or specific plans if developers make side agreements with the Sanitation Districts? 6

The agreement between the developer of the Newhall Ranch Project and the Sanitation District violates the conditions of the Newhall Ranch Specific Plan and puts the Santa Clarita Valley in jeopardy of continued non-compliance with the Clean Water Act Chloride TMDL. We therefore strongly object to this agreement and ask that the Board of Supervisors take action to rectify this issue. 7

The public should not have to pay the costs of bringing the chloride level into compliance with an increase to their sewer fees. Thank you in advance for addressing these issues. 8

Sincerely,

Lynne Plambeck
President

LYNNE PLAMBECK: GOOD AFTERNOON. WHO WOULD YOU LIKE TO START? OKAY. MY NAME IS LYNN PLAMBECK, AND I'M HERE REPRESENTING SANTA CLARITA ORGANIZATION FOR PLANNING AND THE ENVIRONMENT. AND WE'RE CONCERNED ABOUT THIS AGENDA ITEM NOT FOR THE RESOLUTION ITSELF, WHICH IS PERFECTLY CORRECT, BUT FOR THE STATEMENT WITHIN THE RESOLUTION THAT SAYS THAT "THE FIRST 6,000 UNITS OF NEWHALL RANCH WILL BE PUT THROUGH THE VALENCIA TREATMENT PLANT." THAT'S NOT CONSISTENT WITH THE NEWHALL RANCH THAT WAS PASSED FOR THE FORMATION OF THIS, THE NEWHALL RANCH SANITATION PLANT. THE NEWHALL RANCH SANITATION PLANT IS SUPPOSED TO BE A REVERSE OSMOSIS PLANT, WHICH WILL HELP HANDLE THE CHLORIDE PROBLEM WHICH MAYOR ANTONOVICH, I KNOW YOU'RE WELL AWARE OF, AS YOU SIT ON OUR SANITATION DISTRICT AND HAD TO GO THROUGH ALL THOSE HEARINGS. SO OUR OBJECTION IS NOT TO THE RESOLUTION ITSELF, BUT TO THE FACT IN THE STAFF REPORT THAT SAYS THOSE FIRST 6,000 UNITS WILL GO THROUGH THE EXISTING VALENCIA TREATMENT PLANT, OR UP TO 6,000 UNITS. AND WE ASK THAT THAT BE STRUCK FROM THE STAFF REPORT BECAUSE IT SEEMS TO BE A BACKDOOR WAY OF GETTING THOSE APPROVED WHEN THERE'S NO E.I.R. ON THAT AND IT'S NOT CONSISTENT WITH THE SPECIFIC PLAN. THE ADDENDUM ITSELF THAT WAS PASSED FOR THE FORMATION IN 2005 AT L.A.F.C.O. FOR THE FORMATION OF THE SANITATION DISTRICT SPECIFICALLY SAYS THAT THE WASTEWATER TREATMENT PLANT WILL BE BUILT IN STAGES AS THE SPECIFIC PLAN AREA IS DEVELOPED AND WILL ULTIMATELY BE SIZED TO TREAT UP TO 6.8 MILLION GALLONS.

SO IT, TOO, IS NOT CONSISTENT WITH WHAT IS BEING SAID IN THE STAFF REPORT. SO WE WONDER HOW THE SANITATION DISTRICT WOULD HAVE MADE AN AGREEMENT LIKE THAT THAT'S IN VIOLATION OF YOUR ENVIRONMENTAL DOCUMENTS AND THE SPECIFIC PLAN. SO WE ASK YOU TO INVESTIGATE THAT. AND THEN FURTHER WE ASK THAT THIS INFORMATION THAT 6,000 UNITS IS GOING TO GO THROUGH THE VALENCIA TREATMENT PLANT BE STRUCK FROM THE STAFF REPORT. BECAUSE THE NEXT THING THAT WILL HAPPEN IS IT WILL SHOW UP IN THE TRACT MAPS AS THOUGH YOU HAD APPROVED IT. WE DID SEND LETTERS TO EACH SUPERVISOR'S OFFICE. WE'D LIKE TO RESUBMIT THOSE LETTERS AT THIS TIME. THANK YOU. ONLY ENOUGH FOR THE SUPERVISORS. I'M SORRY I DIDN'T PROVIDE IT FOR YOU. THAT'S ALL I HAVE TO SAY, EVEN THOUGH I HAVE 35 SECONDS LEFT.

SUP. ANTONOVICH, MAYOR: THANK YOU, LYNNE, GOOD JOKE.

CAM NOLTEMEYER: MY NAME IS CAM NOLTEMEYER, ALSO SPEAKING FOR SANTA CLARITA ORGANIZATION FOR PLANNING AND THE ENVIRONMENT. MAYOR ANTONOVICH, I KNOW YOU WERE AT A MEETING IN SANTA CLARITA ALONG WITH A COUPLE OF OUR COUNCIL MEMBERS WITH REGARD TO THE SANITATION DISTRICT TRYING TO PUT UPON THE TAXPAYERS 250 MILLION TO 500 MILLION FOR THE COST OF A REVERSE OSMOSIS TREATMENT PLANT TO REMOVE CHLORIDES TO MEET THE CLEAN WATER STANDARD. BECAUSE OF THAT, THERE WAS A LOT OF PUBLIC PROTEST, AND THAT MEETING DECISION HAS BEEN POSTPONED. AT THAT TIME,

THEY NOT ONLY WERE OBJECTING TO THE COST OF 250 TO 500 MILLION, BUT THEY WERE ALSO CONCERNED ABOUT THE FACT THAT THEY WERE FUNDING THE SANITATION DISTRICT FOR NEWHALL RANCH. AT THAT TIME, THE SANITATION DISTRICT MADE IT VERY CLEAR THAT THAT WOULDN'T HAPPEN. NOW WE APPEAR BEFORE YOU, AND NEWHALL LAND IS CLAIMING THAT THEY HAVE THIS AGREEMENT WITH THE SANITATION -- ACTUALLY SANTA CLARITA VALLEY SANITATION DISTRICT OF LOS ANGELES COUNTY TO ALLOW THESE 6,000 UNITS TO BE TREATED IN OUR EXISTING SANTA CLARITA WASTEWATER FACILITIES. THOSE FACILITIES ARE NOT REVERSE OSMOSIS PLANTS. AND IF THIS IS ALLOWED, IT WILL ONLY CREATE ADDITIONAL PROBLEM AS FAR AS THE CHLORIDES FOR OUR COMMUNITY. THE REVERSE OSMOSIS PLANT THAT IS REQUIRED WITH THIS NEWHALL TREATMENT PLANT, THAT WILL TAKE CARE OF CHLORIDES. SO DEFINITELY, THEY SHOULDN'T BE ALLOWED TO USE ANY OTHER TREATMENT PLANT. THEY CAN BUILD THIS PLANT, AND IT'S QUITE CLEAR IN THE DOCUMENTS WE HAVE HERE THAT THEY CAN BUILD THAT IN STAGES IN THE SAME WAY THEY ARE PLANNING TO BUILD THE HOUSES IN STAGES. SO WE DEFINITELY ARE OBJECTING TO ANY IDEA OF 6,000 UNITS OF NEWHALL RANCH BEING ALLOWED TO USE THE CURRENT SANITATION FACILITIES BECAUSE OF THE CHLORIDE ISSUE. AND IT'S A VERY, VERY EXPENSIVE ISSUE FOR OUR COMMUNITY. AND WE WERE PROMISED THAT WE WOULD NOT BE FUNDING ANYTHING FOR THE NEWHALL RANCH. I HAVE 31 SECONDS, BUT THAT IS ALL I HAVE FOR TODAY, THANK YOU.

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LYNNE PLAMBECK: AND IF THAT'S WHAT THEY'RE GOING TO DO, THEY HAVE TO HAVE ADDITIONAL ENVIRONMENTAL ANALYSIS ON IT.

SUP. ANTONOVICH, MAYOR: OKAY.

MARK DILLON: MARK DILLON. GOOD AFTERNOON, SUPERVISORS. HERE SIMPLY TO SUPPORT YOUR STAFF AND THE ACTION HERE TODAY THAT IS BEFORE YOU. AND IF THERE ARE ANY QUESTIONS, I'M HERE TO ANSWER THOSE.

SUP. ANTONOVICH, MAYOR: OKAY, THANK YOU. THANK YOU. LET ME ASK DEPARTMENT OF PUBLIC WORKS? COULD YOU RESPOND TO THE NUMBER OF ISSUES THAT HAVE BEEN RAISED RELATIVE TO THIS PROPOSAL?

SPEAKER: TOM LE BRUN WITH THE SANITATION DISTRICT IS HERE WITH ME. HE MAY BE ABLE TO ANSWER THE QUESTIONS REGARDING THE CONCERNS REGARDING THE 6,000 UNITS.

TOM LEBRUN: THE SANTA CLARITA VALLEY BOARD APPROVED AN AGREEMENT WITH NEWHALL LAND AND FARMING IN 2002 WHICH ALLOWED UP TO 6,000 OF THE FIRST HOMES OF THE NEWHALL RANCH DEVELOPMENT TO GO THROUGH THE VALENCIA W.R.P. BUT IT REQUIRES THE NEWHALL COMPANY TO PAY THE SAME RATES THAT ANY OTHER DEVELOPER WOULD HAVE IN SANTA CLARITA SANTITATION DISTRICT, THAT BEING A CONNECTIONS FEE TO BUY INTO THE EXISTING SYSTEM,

AND THEN AN ANNUAL PAYMENT FOR THE COST TO OPERATE AND TREAT THE 6,000, THE WASTE FROM THE 6,000 HOMES. SO THAT AGREEMENT PREDATED SOME OF THE OTHER AGREEMENTS AND ACTIONS THAT THE BOARD AND L.A.F.C.O. HAVE TAKEN PREVIOUSLY. AND I THINK THAT WAS THE ONE QUESTION ABOUT WOULD THERE BE FUNDING BY SANTA CLARITA VALLEY RESIDENTS FOR NEWHALL RANCH? THE ANSWER IS NO. NEWHALL LAND AND FARMING WOULD HAVE TO PAY, JUST LIKE ANY OTHER DEVELOPMENT, TO USE THE VALENCIA W.R.P. I DON'T KNOW IF THERE WAS ANY OTHER SPECIFIC QUESTIONS ON --

SUP. ANTONOVICH, MAYOR: SO WHO PAYS FOR THE -- WHEN IT HAS TO BE ENLARGED, WHO PAYS FOR IT?

TOM LE BRUN: WHEN THE VALENCIA PLANT WOULD BE ENLARGED --

SUP. ANTONOVICH, MAYOR: THE NEW PROPOSAL, THE NEW DEVELOPMENT, IS THAT GOING TO HAVE A BUILT-IN FEE FOR EXPANSION OF THAT UNIT?

TOM LE BRUN: THE 6,000 HOMES WOULD USE UP SOME AVAILABLE CAPACITY IN SANTA CLARITA VALLEY. THIS IS A TEMPORARY ARRANGEMENT UNTIL THE NEWHALL RANCH W.R.P. IS CONSTRUCTED. THAT WOULD BE PAID FOR BY NEWHALL LAND AND FARMING. AND THEN THE FLOWS WOULD, INSTEAD OF BEING PUMPED TO VALENCIA, WOULD FLOW BY GRAVITY TO THE NEW TREATMENT PLANT AT THE COUNTY LINE.

SUP. ANTONOVICH, MAYOR: SO THEIR ADDITION IS NOT GOING TO MAXIMIZE THE CAPACITY, IT'S GOING TO USE EXISTING RESERVE CAPACITY SO WE DON'T HAVE TO BUILD A FUTURE ONE?

TOM LE BRUN: THAT IS CORRECT.

SUP. ANTONOVICH, MAYOR: AND THEY'RE GOING TO PAY A FEE TO USE THE EXISTING FACILITY WHILE THE NEW ONE'S BEING CONSTRUCTED?

TOM LE BRUN: THAT IS CORRECT.

SUP. ANTONOVICH, MAYOR: AND THE EXISTING FEE WILL PAY FOR ALL MAINTENANCE?

TOM LE BRUN: YES.

SUP. ANTONOVICH, MAYOR: IT WILL COVER THE MAINTENANCE.

TOM LE BRUN: THEY WILL PAY A FEE TO CONNECT, WHICH IS THE SAME THAT ANY DEVELOPER WOULD PAY ANYWHERE IN SANTA CLARITA VALLEY SANITATION DISTRICT, AND THEY WOULD HAVE TO PAY EVERY YEAR THE COST TO TREAT THE WASTE FROM THOSE 6,000 HOMES.

SUP. ANTONOVICH, MAYOR: WHAT ABOUT THE OTHER ISSUES THAT WERE RAISED?

TOM LE BRUN: I AM UNABLE TO TELL YOU BY MEMORY WHAT C.E.Q.A. COVERAGE THERE WAS IN 2002 WHEN THE AGREEMENT WAS PASSED. I JUST DO NOT HAVE THE ANSWER TO THAT. AND IN TERMS OF THE SPECIFIC COVERAGE IN THE NEWHALL SPECIFIC PLAN WITH THE ABILITY FOR THIS WASTEWATER TO TEMPORARILY GO TO THE VALENCIA PLANT, I AM UNAWARE OF THE DETAILS OF HOW THAT ISSUE'S COVERED IN THE NEWHALL RANCH SPECIFIC PLAN, THE E.I.R.

SUP. ANTONOVICH, MAYOR: COULD YOU ALSO REPORT TO THE BOARD ON THE POINTS THAT WERE RAISED AND THEIR TESTIMONY SO WE COULD HAVE THAT?

TOM LE BRUN: I'M SORRY, I DIDN'T HEAR YOU, SUPERVISOR.

SUP. ANTONOVICH, MAYOR: I WOULD LIKE YOU TO RESPOND TO THE BOARD IN A MEMO THE POINTS THAT WERE RAISED BY THE THREE SPEAKERS ON THIS ISSUE.

TOM LE BRUN: CERTAINLY.

LYNNE PLAMBECK: MR. MAYOR, COULD YOU SPECIFICALLY ASK HIM TO RESPOND TO WHO IS GOING TO PAY FOR THE COST OF THE CHLORIDES IN THE WATER? BECAUSE THERE IS NO REVERSE OSMOSIS.

SUP. ANTONOVICH, MAYOR: THAT'S ONE OF THE QUESTIONS THAT YOU RAISED, AND THAT WILL BE INCLUDED IN THAT REPORT.

LYNNE PLAMBECK: THANK YOU VERY MUCH.

SUP. ANTONOVICH, MAYOR: OKAY, THANK YOU. THANK YOU, THANK YOU. SO MOVED. SECONDED BY SUPERVISOR RIDLEY-THOMAS. WITHOUT OBJECTION, SO ORDERED. ITEM NO. 5. DAVID CZAMANSKE, LAURIE GOULD, CAMRON STONE, GLEN OWENS, AND CAROL, YOU'LL FOLLOW AFTER THE FIRST SPEAKER, OKAY? THAT'S THE ARTICLE FROM THE "WALL STREET JOURNAL." SO THE FIVE OF YOU ARE STILL HERE, RIGHT? COME ON UP AND TAKE YOUR SEAT. OKAY. YOU CAN GO IN THE ORDER YOU WANT.

LAURIE GOULD: I'M SPEAKING TO THE ESTABLISH A WORKING GROUP WITH COMMUNITY REPRESENTATIVES TO PROVIDE INPUT TO THE LOS ANGELES --

SUP. ANTONOVICH, MAYOR: GIVE YOUR NAME FOR THE RECORD, FIRST SO THAT WE CAN IDENTIFY YOU.

Prepared for

Newhall Land
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Valencia, California 91355-1088

LANDMARK VILLAGE
WATER QUALITY TECHNICAL REPORT

Prepared by

Geosyntec consultants

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Project Number WW1349

September 2011

Landmark Village
Water Quality Technical Report

Prepared for

Newhall Land
Valencia, California

Prepared by

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Oakland, California

September 2011

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Acronyms

ASCE: American Society of Civil Engineers
BAT: Best Available Technology
BCT: Best Conventional Technology
BMP: Best Management Practice
BOD: Biochemical Oxygen Demand
Cd: Cadmium
CACO3: Calcium Carbonate
CASQA: California Stormwater Quality Association
CDFG: California Department of Fish and Game
CEQA: California Environmental Quality Act
CLWA: Castaic Lake Water Agency
CNLM: Center for Natural Lands Management
CTR: California Toxics Rule

Cu:	Copper
CWA:	Clean Water Act
CWP:	Center For Watershed Protection
DCIA:	Directly Connected Impervious Area
DDT:	Dichlorodiphenyltrichloroethane
DLWC:	Department of Land and Water Conservation
EIR:	Environmental Impact Report
EMC:	Event Mean Concentration
Ep:	Erosion Potential
ETo:	Reference Evapotranspiration Rate
FIB:	Fecal Indicator Bacteria
HOA:	Home Owners Association
IPM:	Integrated Pest Management
JPA:	Joint Powers Authority
LACDPW:	Los Angeles County Department of Public Works
LARWQCB:	Los Angeles Regional Water Control Board
LID:	Low Impact Development
LMD:	Local Maintenance District
LSAA:	Lake and Streambed Alteration Agreement
MBAS:	Methylene Blue Activated Substances
MEP:	Maximum Extent Practicable
MGD:	Million Gallons per Day
mg/L:	Milligrams per Liter
MPN:	Most Probable Number
MS4:	Municipal Separate Storm Sewer System
NCDC:	National Climatic Data Center
NPDES:	National Pollutant Discharge Elimination System
NRSP :	Newhall Ranch Specific Plan
NSWQ:	National Storm Water Quality Database
O&M:	Operations and Maintenance
PAH:	Polycyclic Aromatic Hydrocarbons
Pb:	Lead
PCB:	Polychlorinated Biphenyl
PDF:	Project Design Feature
PIPP:	Public Information and Participation Program
RMDP:	Resource Management and Development Plan
RWQCB:	Regional Water Quality Control Board
SAR:	Sodium Adsorption Ratio
SCR:	Santa Clara River
SEA:	Significant Ecological Area
SMA:	Special Management Area
SQMP:	Stormwater Quality Management Plan

SWMP:	Stormwater Mitigation Plan
SUSMP:	Standard Urban Stormwater Mitigation Plan
SWPPP:	Storm Water Pollution Prevention Plan
SWRCB:	State Water Resources Control Board
TDS:	Total Dissolved Solids
TKN:	Total Kjeldahl Nitrogen
TMDL:	Total Maximum Daily Load
TN:	Total Nitrogen
TP:	Total Phosphorus
TPH:	Total Petroleum Hydrocarbons
TRM:	Turf Reinforcement Mat
TSS:	Total Suspended Solids
USEPA:	United States Environmental Protection Agency
USGS:	United States Geological Survey
WEF:	Water Environment Federation
WDR:	Waste Discharge Requirements
WMA:	Watershed Management Area
WMC:	Watershed Management Committee
WRP:	Water Reclamation Plant
WQTR:	Water Quality Technical Report
Zn:	Zinc

1 INTRODUCTION

This revised report, September 2011, addresses the potential impacts of the proposed Landmark Village Project (the Project), a portion of the Newhall Ranch Specific Plan, on water quality in the Project's receiving waters, the Santa Clara River. To evaluate impacts of the Project on water quality, pollutants of concern are identified based on regulatory and other considerations. Potential changes in water quality are addressed for pollutants of concern based on runoff water quality modeling, literature information, and professional judgment. Impacts take into account Project Design Features (PDFs) selected consistent with the Los Angeles County Municipal Separate Storm Sewer System (MS4) National Pollutant Discharge Elimination System (NPDES) Permit, including the Standard Urban Stormwater Mitigation Plan (SUSMP) requirements. The level of significance of impacts is evaluated using a weight of evidence approach considering significance criteria that include predicted runoff quality for proposed versus existing conditions, MS4 Permit and Construction General Permit requirements, and reference to receiving water quality benchmarks, including Total Maximum Daily Load (TMDL) wasteload allocations and water quality standards from the Basin Plan and California Toxics Rule.

This Landmark Village Water Quality Technical Report (WQTR) has been revised to, among other things, reflect the following LID Performance Standard, which has been adopted for the Newhall Ranch Specific Plan subregion:

> LID PDFs will be selected and sized to retain the volume of stormwater runoff produced from a 0.75 inch storm event to reduce the percentage of Effective Impervious Area (EIA) to 5 percent or less of the total project area within the Newhall Ranch Specific Plan. Runoff from all EIA will be treated with effective treatment control measures that are selected to address the pollutants of concern and are sized to capture and treat 80 percent of the average annual runoff volume. Compliance with the LID Performance Standard will be evaluated for each village within the Resource Management and Development Plan (RMDP) as part of the Tier 2 Water Quality Technical Report process. Each Tier 2 project must demonstrate that the LID Performance Standard is achieved cumulatively considering the retention volume provided by the project itself and all previous projects within the RMDP area.

Further detail on implementation of the LID Performance Standard is provided in Section 5.2 of this WQTR.

The report also assesses the potential for post-development stormwater runoff discharge rates, velocities, and durations to cause accelerated stream erosion and to impact stream habitat, and includes PDFs to address these potential impacts.

The purpose of this WQTR is to assess the Project's potential impacts on surface water and groundwater quality and hydrology in the receiving surface waters and to identify Project Design

Features for inclusion in the Project. Geosyntec Consultants has prepared a Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan (NRSP Sub-Regional SWMP) (Geosyntec, 2008). This Landmark Village WQTR is an independent component of, and is consistent with, the framework for stormwater water quality and hydromodification management established by the NRSP Sub-Regional SWMP, serves as the Tier 2 project report for Landmark Village, and shows compliance with the tiered planning process established in the NRSP Sub-Regional SWMP.

The U.S. Army Corps of Engineers (Corps) and the California Department of Fish and Game (CDFG), as lead agencies, have prepared a joint Final Environmental Impact Statement/ Environmental Impact Report (EIS/EIR) for the Newhall Ranch Resource Management and Development Plan (RMDP) (SCH No. 200001125) (Corps and CDFG, 2010). The Newhall Ranch RMDP consists of those measures and project design features necessary to avoid, minimize and mitigate the adverse biological effects of improvements, facilities, and activities associated with implementation of the Newhall Ranch Specific Plan that will require federal and state permits and agreements from the Corps and CDFG. Essentially, the RMDP is the biological mitigation program for the Newhall Ranch Specific Plan. The RMDP has been prepared to plan, define, and govern the implementation of various avoidance, minimization and mitigation measures required for implementation of the Newhall Ranch Specific Plan, including onsite and offsite drainage facilities, bridges, building pads, roads, trails, and facilities associated with the approved Newhall Ranch WRP.

Potential hydrologic impacts related to stormwater volume and velocity from the 50 year storm event and the 50 year Capital Flood event are addressed in “Landmark Village Tentative Tract Map 53108 Drainage Concept” prepared by Psomas (Psomas, 2009) and the “Flood Technical Report” prepared by Pacific Advanced Civil Engineering, Inc. (PACE) for the Landmark Village Project (PACE, 2009). Potential biological impacts of the Landmark Village Project are addressed in the Landmark Village Final EIR, prepared by Impact Sciences, Inc. (Impact Sciences, 2011). An engineering analysis of streambed fluvial stability in the Santa Clara River also has been prepared by PACE (PACE, 2006).

The approved Newhall Ranch Specific Plan was the subject of extensive environmental review in the previously certified Newhall Ranch Program EIR (State Clearinghouse No. 1995011015) and related Revised Additional Analysis, Volume VIII (May 2003). The Specific Plan was assessed at the program level as part of the environmental analysis conducted for the Newhall Ranch Specific Plan. Portions of that analysis, including the certified Flood Section (Section 4.2 – Newhall Ranch Specific Plan EIR) and the certified Revised Additional Analysis (Section 2.3, Floodplain Modifications), have been used in the development of this Project WQTR.

2 ENVIRONMENTAL SETTING

2.1 Physical Setting

The Project site is located within the Newhall Ranch Specific Plan area, which is in an unincorporated portion of Los Angeles County, approximately 30 miles northwest of downtown Los Angeles. The site is in the Santa Clarita Valley, west of Interstate 5. The developed portion of the Project (tract map site) lies between the banks of the Santa Clara River to the south, SR-126 to the north, the confluence of Castaic Creek and the Santa Clara River to the east, and Chiquita Creek to the west (Figure 2-1). For the purposes of this report, the "Project developed area" refers to the proposed location of the Landmark Village development, while the "Project site" includes the tract map site, the borrow site and related haul routes, the Chiquito Canyon grading site, the utility corridor, and the potable and recycled water tank sites.

The Project impact boundary depicted on Figure 2-1 includes the developed portion of the Project (Landmark Village tract map site), as well as areas that will be temporarily disturbed during the construction phase of the Project, which includes the borrow site and other areas of grading, and areas where underground utilities will be installed.

The proposed tract map site lies on a flat terrace above the Santa Clara River. The majority of the tract map site is currently used for agricultural purposes and is subject to agricultural disking. Topography across the site is relatively flat, with elevations ranging from 800 feet to 960 feet above mean sea level (msl). Habitat on the Project site varies in quality from high biological value in riparian areas adjacent to the Santa Clara River channel, to highly disturbed habitat such as upland agricultural areas. According to the Antelope Valley Area Soil Survey (Soil Conservation Service 1970), nine soil types occur on the Project site: Sandy alluvial land, Metz sandy loam, Hanford sandy loam (0 to 2%), Hanford sandy loam (2 to 9%), Sorrento loam (0 to 2%), River wash, Saugus loam (30 to 50%), Castaic and Saugus soils, and Zamora loam.

Fill has been placed on the tract map site as a result of road construction, utility line placement, and agricultural activities. Fill also exists at various locations on both borrow sites, ranging from minor spill fills to large dumped fill pads associated with historical uses.

The proposed borrow site is characterized by sloping hillsides and adjacent agricultural use. The borrow site is dominated by coastal sage scrub and mixed chaparral, but also includes several small areas of non-native grassland and live oak woodland. Elevations on the borrow site range from approximately 920 feet (near the Santa Clara River) to 1260 feet above msl. The Chiquito Canyon grading site is dominated by agricultural/disturbed areas, non-native grassland and coastal sage scrub vegetation. Elevations at this grading site range from approximately 970 feet (near SR-126) to 1,190 feet above msl.

The drainage area that encompasses the tract map site consists of six sub-basins that independently drain toward the Santa Clara River (Psomas, 2009). There are currently no existing drainage or erosion/sedimentation control improvements located within the site other than minor agricultural drainage ditches and an insignificant amount of earthen bank protection along the Santa Clara River (PACE, 2009). A jurisdictional delineation of waters and streambeds was conducted in accordance with the Corps protocol in 2003 (Impact Sciences). The tract map site is generally bordered to the east by Castaic Creek, to the south by the Santa Clara River and to the west by Chiquito Canyon Creek. These drainages are under the jurisdiction of the Corps. The Chiquita Landfill area drains through an agricultural drain located in the central portion of the tract map site that is also under the jurisdiction of the Corps. There are no other drainage features within the Project boundaries that are under the jurisdiction of the Corps.

The Project lies downstream from two water reclamation plants. The Saugus Water Reclamation Plant (WRP) is located 5 miles upstream from the Project, across Bouquet Canyon Road at Soledad Canyon Road, and the Valencia Water Reclamation Plant is located 1.5 miles upstream, just north of Magic Mountain Parkway at the Old Road. Both treatment plants discharge treated wastewater into the Santa Clara River upstream from the Project.

2.2 Project Area Land Uses

The Project site is located within the Newhall Ranch Specific Plan (NRSP) area, which was approved by the Los Angeles County Board of Supervisors in May 2003. The Newhall Ranch Specific Plan is a comprehensive document that guides future development of the Newhall Ranch property and serves as the zoning for the entire Specific Plan area. The Specific Plan contains a conceptual development plan, development regulations, design guidelines, and implementation mechanisms consistent with the goals, objectives, and policies of the Los Angeles County General Plan and Santa Clarita Area Plan. The NRSP is a large, master-planned development including approximately 21,000 homes and 19,000 jobs, along with recreational and mixed uses and public facilities. A complete description of the land uses included in the NRSP can be found in the Newhall Ranch Specific Plan (SCH # 1995011015; May 2003).

The proposed Project is to develop the 292.6-acre Landmark Village tract map site, located within the boundary of the approved Newhall Ranch Specific Plan. To facilitate development of the Landmark Village tract map site, several off-site project-related components would be developed on an additional 750.9 acres of land that, for the most part, is within the approved Specific Plan boundary.[1] The land uses proposed as part of the project are consistent with the Newhall Ranch Specific Plan. The Land Use Plan contained within the NRSP designates the Landmark Village Project for single and multi-family residential, commercial, and recreational

[1] Portions of the proposed utility corridor, the proposed potable water tank site (located within the Valencia Commerce Center business park) and the proposed recycled water tank (located within the Valencia Industrial Center business park) are outside the boundary of the Newhall Ranch Specific Plan.

land uses. Existing land use in the developed portion of the Project area consists completely of agricultural production.

In response to comments received on the Landmark Village Recirculated Draft EIR (RDEIR) from CDFG, the County has directed the applicant to submit a revised Vesting Tentative Tract Map (revised project) that, among other design components, reflects an additional riparian buffer, or setback, that reduces impacts to riparian resources that fall within CDFG's jurisdiction. In response to the County's directive, the applicant has designed a revised project. Specific to CDFG's comments, the setback proposed by the applicant as part of its revised project occur along the west bank of Castaic Creek between SR-126 and the confluence of Castaic Creek and the Santa Clara River, and along the northern and southern banks of the Santa Clara River.

Key changes to the proposed project resulting from refinements made in response to CDFG's comments and at the County's direction are summarized as follows:

- Development Footprint: The size of the development footprint on the project tract map site would decrease by 16.9 acres (a 7 percent decrease in the development footprint).
- Bank Stabilization and Outlets: The linear feet of buried bank stabilization along portions of the Santa Clara River would decrease by 357 feet, from 18,600 to 18,243 feet. The number of outlets to the Santa Clara River would decrease from 13 to 9.
- Open Space: Within the open space project component, the total amount of open space would increase from approximately 65 to 76.7 acres, for an overall increase of 11.7 acres.

Other changes resulting from the key revisions to the proposed Project include the following:

- Residential: The total number of residential dwelling units would remain unchanged at 1,444 units. Within the residential project component, however, the number of single-family units would decrease from 308 to 270 units (a reduction of 38 units); the number of multi-family units would increase from 1,080 to 1,105 units (an increase of 25 units); and the number of mixed-use/multi-family units would increase from 56 to 69 units (an increase of 13 units). The range of single-family lot sizes would change from 4,500/5,500/6,000 square feet to 4,500/5,000/5,500 square feet, and the average density of residential development would change as follows: single-family density would increase from 6.3 to 7.3 units per acre (du/acre), multi-family density would decrease from 14.6 to 14.0 du/acre, and mixed-use/multi-family would increase from 9.5 to 16.8 du/acre.
- Mixed-Use/Commercial: While the total square footage of commercial space would remain unchanged at 1,033,000 square feet, the acres of mixed-use commercial area would decrease from 33.9 to 25.1 acres (a decrease of 8.8 acres, or 26 percent), and the average floor-to-area ratio (FAR) would increase from 0.7 to 0.9 (an increase of 0.2).

- Elementary School: The size of the elementary school lot would increase from 9.2 to 9.7 acres.
- Park and Ride: The size of the park-and-ride lot would decrease from 1.0 to 0.8 acres. [2]

The land use areas analyzed in this revised WQTR, which reflect the revised project design, are listed in Table 2-1 below and illustrated in Figure 2-2.

Table 2-1: Summary of Revised Land Use Program Analyzed

Land Use Designation	Landmark Village Project (Acres)
Single-Family[1]	53.9
Multi-Family	82.9
Commercial[2]	27.3
School	9.7
Road[3]	41.6
Open Space	51.2[4]
Park	10.1
Recreation	5.8
Water Quality Facility	10.1
Total	*292.6*
Off-Site Commercial (Water Tanks)	8.0
Off-Site Road[5]	98.0
Total Area	398.6

1 16.7 acres of residential roads are included in the Single-Family land use category.
2 Commercial land use acreage includes Mixed-Use/Commercial, Fire Station, and Park and Ride land uses.
3 Road land use includes minor roads (private drives and access road) and major roads, and excludes 16.7 acres included in Single-Family, and 0.7 acres of Light Rail Reservation, which is included in Open Space.
4 18.5 acres of open space were not included in the water quality model.
5 Off-site roads consist of 2.4 acres of off-site bridge to the south and 95.6 acres of SR-126 right of way to the north.

The two sources of non-potable supplies needed to meet the Project's non-potable water demand (e.g., irrigation) are recycled water from the planned Newhall Ranch WRP and from the existing Valencia WRP. The Newhall Ranch WRP's treatment capacity is planned to be 6.8 mgd of wastewater generated by the Specific Plan, all of which ultimately would be treated at the Newhall Ranch WRP, and upon tertiary treatment, recycled for landscape irrigation purposes (except for wet winters periods when irrigation demands would be lower, requiring the discharge of unused recycled water to the Santa Clara River).

The Newhall Ranch WRP, to be located along the Santa Clara River in the western edge of the Specific Plan site, is planned to be constructed in stages as the Specific Plan is developed over time. In the interim, the Santa Clarita Valley Sanitation District (SCVSD) will treat the

2 The revised VTTM would not change the size of the proposed fire station lot; it would remain 1.3 acres in size.

wastewater from the first 6,000 residential dwelling units within the Specific Plan (up to 1.6 mgd) at the existing Valencia WRP as needed prior to construction/operation of the first phase of the Newhall Ranch WRP. The interim use of the Valencia WRP is contemplated by the 2002 Interconnection Agreement entered into between Newhall and the Sanitation District Nos. 26 and 32 (later consolidated as the SCVSD).

As proposed in the interim period, treated effluent from the Valencia WRP would be piped to the proposed chloride reduction facility adjacent to the Valencia WRP for demineralization (using reverse osmosis or equivalent). Treated effluent would be piped back to the Valencia WRP and blended with treated effluent so that up to approximately 6,000 units (approximately 1.6 mgd) of effluent generated by Newhall Ranch Specific Plan in the interim condition would be discharged at less than 100 mg/L for chloride. The brine by-product of the chloride reduction process would be piped within the Project's utility corridor north along The Old Road, west on Henry Mayo Drive and north on Commerce Center Drive to the brine disposal facility, located in the Valencia Commerce Center, just north of Castaic Creek. The piping necessary north of the utility corridor along Commerce Center Drive also would be installed within the existing roadway. The brine would be injected deep into the Pliocene Pico and Miocene Modelo formations at this location (the location of these facilities is shown on Figure 2-1). Piping needed to transport effluent from the demineralization facility to the injection wells would be sized to the satisfaction of the Sanitation District. The applicant has applied to the U.S. Environmental Protection Agency (USEPA) for approval to construct the brine injection facility.

The Newhall Ranch Revised Draft EIR (March 8, 1999) contains a project-level analysis of the potential significant environmental impacts associated with construction and operation of the approved Newhall Ranch WRP. This Landmark Village WQTR addresses the potential impacts of recycled water use for irrigation on groundwater quality and considers the potential cumulative impacts of WRP discharges on water quality and hydromodification in the Santa Clara River.

2.3 Associated Off-Site Project Components

In addition to the 292.6-acre tract map site, the Project also includes grading and/or development at off-site project locations.

2.3.1 Long Canyon Road Bridge

The Project includes construction of the Long Canyon Road Bridge. The bridge is intended as the primary bridge crossing over the Santa Clara River providing access to the central portions of the Newhall Ranch Specific Plan. The bridge will span approximately 1,050 feet over the Santa Clara River, with a width of approximately 100 feet. Support for the bridge will involve construction of 11 piers within the river corridor. Each pier will be spaced approximately 100 feet apart. Additionally, abutments and bank stabilization would be required on both sides of the bridge to protect against erosive forces.

2.3.2 Borrow Site

Site preparation would include a cut and fill grading operation with fill being imported to the tract map site from a borrow site located south of the tract map site. To elevate the Project development above the floodplain of the Santa Clara River, soil would be imported from a borrow site located within Adobe Canyon (Figure 2-1). The borrow site is located south of the Santa Clara River and is bounded by Long Canyon to the west. The total drainage area for the site is about 215 acres and flows generally northwest and westerly. The majority of the land is undeveloped with steep to moderate slopes.

The borrow site grading plan would excavate and reshape the hills and depressions forming the ridge separating Long and Adobe Canyons. Much of this work would occur along the top and bluffs of an unnamed plateau located just west of Sawtooth Ridge. This plateau ranges in elevation from a low of 1,130 feet at its northern most point to a high of 1,220 in the southeast, which is characterized by an increasingly steeper grade. The proposed grading plan would excavate the southeastern portion of this plateau, creating a gentler slope leading up to the top of the ridge. The resultant manufactured slope angle would range from 5:1 to 2:1 (horizontal/ vertical). The grading plan also would alter the western facing slope leading up to the plateau, creating a bench separated by two manufactured slopes stepping down the west-facing ridgeline defining Adobe Canyon at a 3:1 grade.

Additional earthwork is planned at the terminus of Adobe Canyon where a series of excavations will result in a manufactured slope approximately 100 feet in height at a relatively uniform 3:1 grade. A series of benches, swales and debris basins will also be constructed to collect, convey and release runoff in a controlled manner. The fill excavated from the Adobe Canyon area would be transported across the Santa Clara River to the tract map site using existing at-grade agricultural crossings. All of this area is within the development footprint approved with the Newhall Ranch Specific Plan.

Also in response to CDFG's comments, the revised project and related grading activities in Adobe Canyon have been modified to avoid impacts to spineflower. Specifically, a minimum of 300-foot buffer area has been established to ensure protection of the spineflower occurrence in the Adobe Canyon area, unless or until that area is authorized for take in the future as part of CDFG's issuance of the incidental take permit for the spineflower. The County will only approve a Landmark Village proposed project that is consistent with the CDFG-approved Final SCP (2010).

2.3.3 Chiquito Canyon Grading Site

To accommodate Project-necessitated improvements to SR-126 and debris basins for storm water flows that are collected by the Project storm drainage system, approximately 120 acres of land directly north of SR -126 and within Chiquito Canyon would be graded. The Conceptual Grading Plan would lower the area of the property near the intersection of Chiquito Canyon

Road with SR-126 by approximately 60 feet when compared to the existing elevation. Rather than a gradual incline that extends upward at an increasingly greater grade, the reshaped slope would approximate the grade of SR-126 for about 1,500 feet east of the intersection with Chiquito Canyon Road. At this point, the grading plan creates a manufactured slope that extends upward at a uniform 3:1 grade reaching a high of 1,160 feet above msl. A series of benches, swales and debris basins will also be constructed to collect, convey and release runoff in a controlled manner. The fill would be excavated from this area and placed as fill in the adjacent areas. All of this graded area is within the development footprint of the NRSP.

2.3.4 Water Tanks

Potable water would be conveyed to the tract map site from two separate water tank sites. One tank is proposed north of the SR-126 within the existing Valencia Commerce Center business park. The second potable water tank would be located within the borrow site, in an area to be graded as part of the proposed soil transfer. The Project would also implement a portion of the Specific Plan's recycled water storage and distribution system with the conversion of an existing potable water tank on Round Mountain (east the tract map site and east of Interstate 5 within the Valencia Industrial Center business park) to a recycled water tank. For the purpose of the impact analysis, the area for the water tanks and access roads to the tanks was estimated to be a total of eight acres.

2.3.5 State Route 126 Improvements

Improvements to State Route 126 (SR-126) would be constructed in conjunction with the project. A 95.6 acre portion of the SR-126 project, extending from just west of the intersection of Commerce Center Drive and SR-126 to the western edge of Landmark Village, including the widening of the Castaic Creek/SR-126 Bridge, has been included in the Project analysis. Along with the bridge deck widening, bridge abutments are to be widened to approximately 500 linear feet (LF) of creek length of reinforced concrete transitioning to soil cement through 50 LF of creek length of rip-rap.

2.3.6 Utility Corridor

The Project also includes a 110-acre utility corridor that runs parallel to SR–126, from the western boundary of the tract map site to the approved Newhall Ranch WRP near the Los Angeles County/Ventura County line, from the eastern boundary of the tract map site to the Old Road and then south to the Valencia WRP. The utility corridor would serve to extend municipal services to the site. The utilities will be placed underground and a maintenance access road and potential future trail would be constructed above ground. As the impact area for the maintenance access road and trail have not yet been determined, the impacts of these Project components are assessed qualitatively in this report.

2.4 Proposed Drainage Improvements – Project and Santa Clara River

The proposed improvements on the Project site that would occur in and adjacent to the Santa Clara River, including bank stabilization, storm drain outfalls and associated energy dissipaters, and construction of Long Canyon Road Bridge across the River are described below. At limited locations on the Project site, such as at outlet structures, access ramps, or bridge abutments, grouted rip-rap or reinforced concrete would be used to minimize erosion. Approximately 18,243 LF of the River and Creek bank would be provided with buried soil cement protection. This would include approximately 10,643 LF fronting the tract map site and 6,400 LF on the south bank downstream (west) of the Long Canyon Road Bridge. Additional buried bank stabilization would be constructed as part of the approved Newhall Ranch WRP and between The Old Road and the Santa Clara River (protecting the utility corridor). The bank protection between the Old Road and the Santa Clara River was approved as part of the Santa Clara River Natural River Management Plan (NRMP). Approximately 6,600 LF of Turf Reinforcement Mat (TRM) or similar bank stability protection would be provide along the southern edge of the utility corridor downstream or west of the tract map site. Additional flood protection improvements would include the Long Canyon Road Bridge abutments (including rip rap) and piers, the widening of the bridge abutments (including rip rap) of the SR 126/Castaic Creek Bridge, and the various outlet structures and energy dissipaters both on-site and off-site. Finally, the above flood control improvements may necessitate the need for dewatering activities. These activities would be subject to the applicable requirements of the LARWQCB.

2.4.1 Proposed Project Drainage Improvements

Runoff from the six off-site drainage areas that drain through or onto the developed portion of the Project site, as defined by the Psomas Landmark Village Drainage Concept Report (Psomas, 2009) would continue to flow through the Project site. Runoff from the developed portions of the Project would be channeled through the proposed stormwater conveyance system and would be discharged to the Santa Clara River through nine new outfalls after passing through the water quality treatment BMPs (see Section 5.3 for further detail). As required in the Los Angeles County Department of Public Works memorandum entitled, “Level of Flood Protection and Drainage Protection Standards,” all on-site drainage systems carrying runoff from developed areas are to be designed for the 25-year Design Storm (Urban Flood), while storm drains under major and secondary highways, open channels (main channels), debris carrying systems, and sumps will be designed for the Capital Flood.

2.4.2 Energy Dissipaters

To reduce storm flow velocities and prevent erosion at stormwater discharge points into the River, energy dissipaters consisting of either rip-rap or other larger reinforced concrete standard impact-type energy dissipaters would be constructed at the approximately nine storm drain outlets into the River, pursuant to the requirements of the Newhall Ranch Resource Management and Development Plan. These energy dissipaters would slow the rate of flow of runoff into the

River to prevent erosion of the stream channel. Additional dissipaters would be located at the outlet of Chiquito Creek and Long Canyon Creek.

2.4.3 Bank Stabilization

The Conceptual Backbone Drainage Plan of the Newhall Ranch Specific Plan provides drainage and flood control protection to developed uses while preserving the Santa Clara River as a natural resource. The Drainage Plan utilizes several criteria that are to be implemented by projects that develop within the Specific Plan area. The primary criteria are as follows:

> Flood corridor must allow for the passage of Los Angeles County Capital Flood discharge without the permanent removal of natural River vegetation (except at bridge crossings);
>
> The bank stabilization for the River will generally be established outside of the "waters of the United States" as defined by federal laws and regulations and as determined by the delineation completed by the Corps in August 1993;
>
> Where the Corps delineation width is insufficient to contain the Capital Flood flow, the flood corridor will be widened by an amount sufficient to carry the Capital Flood flow without the necessity of permanently removing vegetation or significantly increasing velocity; and
>
> Soil cement will occur only where necessary to protect against erosion adjacent to the proposed development. Where existing bluffs are determined to be stable and there is no adjacent proposed development, no bank protection will be built.

Most of the proposed bank protection would consist of buried soil cement to provide scour and freeboard flood control protection. Soil cement bank protection provides a stable riverbank protection material, in terms of both surface erosion and structural stability. Additionally, soil cement bank protection will be mostly buried. After installation, the bank area is revegetated with native vegetation.

2.5 Receiving Water Bodies and Beneficial Uses

2.5.1 Santa Clara River

The Project will discharge from its storm drain and water quality control facilities directly to Santa Clara River Reach 5[3], downstream of its confluence with Castaic Creek.

[3] The SCR is divided into reaches for purposes of establishing beneficial uses and water quality objectives. However, there are two reach classifications, one established by the Los Angeles Regional Water Quality Control Board (LARWQCB) and one established by the United States Environmental Protection Agency (USEPA). Both of these reach classifications are used by the LARWQCB and the USEPA in various documents, which at times is a source of confusion. This report will use the LARWQCB reach numbers.

The Water Quality Control Plan for the Los Angeles Region (Basin Plan) (LARWQCB, 1994, as amended) lists beneficial uses of major water bodies within this region (Table 2-2). Santa Clara River Reach 5 is listed and has specific beneficial uses assigned to it. As identified in Table 2-2, the existing beneficial uses of Santa Clara River Reach 5 include the following:

MUN*: Conditional potential municipal and domestic water supply

IND: Industrial activities that do not depend primarily on water quality

PROC: Industrial activities that depend primarily on water quality

AGR: Agricultural supply waters used for farming, horticulture, or ranching

GWR: Groundwater recharge for natural or artificial recharge of groundwater

FRSH: Natural or artificial maintenance of surface water quantity or quality

REC1: Water contact recreation involving body contact with water and ingestion is reasonably possible

REC2: Non-contact water recreation for activities in proximity to water, but not involving body contact

WARM: Warm freshwater habitat to support warm water ecosystems

WILD: Wildlife habitat waters that support wildlife habitats

RARE: Waters that support rare, threatened, or endangered species and associated habitats

WET: Wetland ecosystems

Table 2-2: Beneficial Uses of Surface Receiving Waters

Water Body	MUN	IND	PROC	AGR	GWR	FRSH	REC1	REC2	WARM	WILD	RARE	WET[1]
Santa Clara River (Hydrologic Unit 403.51)	P*	E	E	E	E	E	E	E	E	E	E	E

[1] Waterbodies designated as WET may have wetlands habitat associated with only a portion of the waterbody. Any regulatory action would require a detailed analysis of the area.

E – Existing beneficial use; P * – Asterixed MUN designations are conditional potential MUN designations[4].

Source: Water Quality Control Plan for the Los Angeles Region (Basin Plan) (LARWQCB, 1994, as amended)

The NRSP area is located within the Santa Clara River Hydrologic Basin and associated watershed, which is 1,634 square miles in area. The portion of the Santa Clara River watershed that is located generally upstream or east of the Ventura County/Los Angeles County jurisdictional line is approximately 640 square miles in size, and drains portions of the Los Padres National Forest from the north, the Angeles National Forest from the north and northeast, and the Santa Susana Mountains from the south and southeast. The Santa Clara River extends approximately 5.5 miles east to west across the NRSP area. The NRSP area comprises 2.9 percent of the Santa Clara River watershed upstream of the Los Angeles County/Ventura County line, 1.1 percent of the total Santa Clara River watershed, and approximately 58 percent of the 20,724-acre tributary drainage area. The Landmark Village project site comprises 972 gross acres within the 1,634 square-mile Santa Clara River Basin watershed.

The Santa Clara River (SCR) watershed drains an area in the Transverse mountain range of southern California. The SCR flows generally west from its headwaters near Acton to the Pacific Ocean near the City of Ventura, approximately 40 miles downstream of the Project location. The river exhibits some perennial flow in its eastern-most stretches within the Angeles National Forest, then flows intermittently westward within Los Angeles County. The principal tributaries of the upper river watershed in Los Angeles County are Castaic Creek, Bouquet Canyon Creek, San Francisquito Creek, and the South Fork of the Santa Clara River. Placerita Creek is a large tributary draining the western-most end of the San Gabriel Mountains; it joins the South Fork, which flows directly into the Santa Clara River. Castaic Creek is a south-trending creek that confluences with the Santa Clara River upstream and adjacent to the Project. Castaic Lake is a DWR-owned reservoir located on Castaic Creek. San Francisquito Canyon Creek is an intermittent stream in the watershed adjacent to Bouquet Canyon to the southeast. Elevations

[4] On December 5, 2001, the U.S. Federal District Court issued an order that effectively invalidated USEPA's requirement that the asterisked MUN designated uses (MUN* uses) in the Los Angeles Basin Plan be immediately enforced. See Order granting plaintiffs' motion for summary judgment and remanding action to USEPA, No. CV 00-08919 R(RZx), *City of Los Angeles et al. v. United States Environmental Protection Agency*, dated December 18, 2001. See also letter dated February 15, 2002, from Alexis Strauss, USEPA Region IX, to Celeste Cantu, Executive Director, California SWRCB: "…waters identified with an ("*") in Table 2-1 do not have an MUN as a designated use until such time as the State undertakes additional study and modifies its Basin Plan." USEPA also stated that this conditional use designation has no legal effect.

within the watershed range from sea level at the river mouth to 8,800 feet at the summit of Mount Pinos in the northwest corner of the watershed.

The principal sources of water contributing to the base flow of the Santa Clara River are: (a) groundwater from the Alluvial aquifer basin in Los Angeles County, which seeps into the riverbed near, and downstream of, Round Mountain (located just below the mouth of San Francisquito Creek); (b) tertiary-treated water discharged to the Santa Clara River from two existing Los Angeles County Sanitation District WRPs -- the Saugus WRP, located near Bouquet Canyon Road bridge, and the Valencia WRP, located immediately downstream of I-5 (for locations, see Figure 2-1); and (c) in some years, DWR-released flood flows from Castaic Lake into Castaic Creek during winter and spring months (CH2M Hill, 2005). The Saugus WRP has a permitted dry weather average design capacity of 6.5 million gallons per day (mgd) creating surface flows from the outfall to near Interstate 5. The Valencia WRP outfall is located immediately downstream of the Interstate 5 bridge and has a permitted dry weather average design capacity of 21.6 mgd, creating surface flows extending through the Project area and into the far eastern portion of Ventura County. The combined average treated discharge from both WRPs between January 2004 and June 2007 was approximately 20 mgd.

The reach of the SCR within and adjacent to the Project has multiple channels (braided). This kind of system is characterized by high sediment loads, high bank erodibility, and intense and intermittent runoff conditions. Combined with the relatively flat gradient of the SCR at this point (less than one percent), the SCR has a high potential to aggrade (deposit sediment) at low flow velocities (PACE, 2006a).

The following description of the physiography, climate, flows, and vegetation of the Santa Clara River are summarized primarily from Assessment of Potential Impacts Resulting from Cumulative Hydromodification Effects, Selected Reaches of the Santa Clara River, Los Angeles County, California (Balance Hydrologics, provided in Appendix F).

Physiography

The Santa Clara River flows through a complex, tectonically-active trough. Some of the most rapid rates of geologically-current uplift in the world are reported from the Ventura anticline and San Gabriel Mountains, just to the northwest and southeast, respectively, of the river. Slopes are very steep, with local relief of 3,000 to 4,000 feet being common. These faults bring harder, more resistant sedimentary rocks over softer and younger sedimentary formations, but all formations are fundamentally soft and erodible. On either side of the faults, sandstone and mudstones prevail. The northeastern and southeastern corners of the watershed are underlain by deeply-weathered granitic and schistose rocks, which produce sands that are coarser than those of other rock units when they weather and erode. The San Gabriel fault crosses the valley, bringing slightly more resistant rock to the surface and creating a local base level reflected as a slight rise or ‘bump’ on the river’s longitudinal profile.

Most geologic materials in the watershed decompose mainly to silts, clays, and sand, with some coarser materials. Most sediment moved by the Santa Clara River and its main tributaries is fine, with less than 5 percent bedload-sized material (>0.25 mm, or about 0.01 inches in diameter). Some gravels and cobbles do occur within the beds of the stream and in their alluvium. Nonetheless, both the bed and the sediment transported by the river tend to be finer than in most Southern California watersheds.

Flows

Downstream of the Valencia WRP, the SCR is perennial past the Los Angeles County/Ventura County line to approximately Rancho Camulos. Flows in the SCR can also be affected by groundwater dewatering operations or by diversions for agriculture or groundwater recharge. Throughout the Santa Clara River channel, there are complex surface water/groundwater interactions where both gaining and losing river segments are found. Downstream of the County line, however, the Santa Clara River flows through the Piru groundwater basin, which represents a "Dry Gap" where dry-season surface flows are interrupted and streamflow is lost to groundwater.

The SCR is underlain by several distinct alluvial groundwater basins in Ventura County—the Piru, Fillmore, and Santa Paula Basins. These basins are divided longitudinally by sills or ridges of bedrock that support areas of locally-high (shallow) groundwater, including the area upstream from the County line (above the Piru Basin), and upstream from the mouth of Sespe Creek (the transition between the Piru and Fillmore Basins). This locally-high groundwater sustains summer baseflow and riparian vegetation within the SCR corridor even through relatively dry climatic cycles.

Flows in the SCR, as in most southern California streams, are highly episodic. For the gaged period between 1953 and 1996, annual flow at the Los Angeles County/Ventura County line gage ranged between 253,000 acre-feet (1969) and 561 acre-feet (1961). Annual peak flows at the County line between 1953 and 1996 ranged from 68,800 cfs (1969) to 109 cfs (1960). Of note is that the second highest annual peak, 32,000 cfs in 1966, was less than half of the highest peak (68,800 in 1969). These large episodic events have a significant impact on the geomorphic characteristics of the Santa Clara River mainstem.

After studying the response of the river to several different anthropogenic and natural disturbances, Balance Hydrologics (2005) concluded that the Santa Clara River, as with many streams in semi-arid southern California, is highly episodic. Concepts of "normal" or "average" sediment-supply and flow conditions have limited value in this "flashy" environment, where episodic storm and wildfire events have enormous influence on sediment and storm flow conditions. In these streams, a large portion of the sediment movement events can occur in a matter of hours or days. Other perturbations which can potentially affect channel geometry appear to have transitory or minor manifestations. For example, effects on SCR channel width due to the 1980s levee construction was barely discernible by the first few years of the 21st

century, probably mostly due to morphologic compensation associated with the storm events in the mid- to late-1990s. As a result, channel morphology, stability, and character of the Santa Clara River is almost entirely determined by the "reset" events that occur within the watershed.

Vegetation and Habitat Types

Much of the watershed upstream of the Newhall Ranch Specific Plan area receives rainfall averaging about 18 to 25 inches per year. As throughout Southern California, rainfall in the Santa Clara watershed alternates between wet and dry periods, a variation that is central to understanding the geomorphic history of the watershed. Wet cycles tend to persist for several years, sometimes for periods of 6 or 8 years, during which rainfall, although variable, may average about 140 to 150 percent of the long-term average. For the woody riparian vegetation along the banks and on islands in the braided channels, these are crucial periods for establishment and growth. During dry cycles, the roots of the riparian vegetation must grow downward to the water table or perched zones, and where it cannot do so, this band of vegetation will die back.

The existing SCR channel contains a variety of vegetation types (Impact Sciences, 2003). The active SCR channel is mostly barren due to scouring by seasonal storm flows. However, vegetation types on the adjacent terraces vary based on elevation relative to the active channel bottom and the frequency of flooding. The following series of vegetation types occur along a vertical gradient from the channel bottom to the highest SCR terrace on the floodplain: emergent herbaceous, woody shrubs, and trees.

The Santa Clara River corridor at the Project site supports three general categories of habitat (Impact Sciences, 2003): (1) aquatic habitats, consisting of flowing or ponded water; (2) wetland habitats, consisting of emergent herbs rooted in ponded water or saturated soils along the margins of the active channel; and (3) riparian habitat, consisting of woody vegetation along the margins of the active channel and on the floodplain. Both year-round and seasonal aquatic habitats are provided and are subject to periodic disturbances from winter flood flows. These flows inundate areas that are dry most of the year. They also carry and deposit sediment, seeds, and organic debris; form new sandbars and destroy old ones; and erode stands of vegetation. New stands of vegetation are created where vegetation becomes established by seeds or buried stems. Thus, the aquatic habitats of the river are in a constant state of creation, development, disturbance, and destruction.

2.5.2 Tributaries to the Santa Clara River

Several tributaries drain into the Santa Clara River adjacent to the Landmark Village Project site, including Chiquito Canyon and Long Canyon, which enter the River downstream of the Project site, and Castaic Creek, which enters the River upstream of the Project site (Figure 2-1). Project runoff from the developed portion of the Project will not be discharged to the tributaries; all Project runoff will be discharged to the Santa Clara River after receiving treatment in the Project

PDFs. Construction phase activities (borrow sources and grading) will occur in areas that drain to Adobe Canyon, Long Canyon, and Chiquito Creek.

The Chiquito Canyon, Long Canyon, and Castaic Creek watersheds are characterized by both rugged and steeply developed foothills that have numerous smaller tributary canyons that dissect the watershed, connecting to the narrow alluvial valley associated with the main stem drainage. Approximately 90 percent or more of the watersheds' area consists of rugged foothill topography with the remainder being the narrow valley floor. Generally, the soils in the watersheds are characterized as silty clay loams from both the Castaic and Saugus formations. Also, the soils within the watersheds can be predominately classified as being in hydrologic soil group C (higher runoff potential) with exception of areas adjacent to the main stem drainages that are Type A (lower runoff potential) and Type B in the lower reaches.

The 4.85 square mile (3,106 acre) Chiquito Canyon watershed is a tributary to the northern bank of the Santa Clara River. Approximately 490 acres of Chiquito Canyon, or only 16% of the watershed area, is located within the NRSP boundary, with the majority being upstream of the NRSP boundary in the developed Val Verde community (PACE, 2006b). The upper portion of the drainage is aligned in a general west to east direction while the lower portion of the drainage flows in a north to south direction. The linear distance from the upper headwaters to the canyon mouth is approximately 28,318 feet, with an average overall slope of 0.031. The major natural main stem drainage course within the watershed has an average slope through the NRSP area of approximately 0.025. The topography for the watershed varies from a maximum elevation of 1,800 feet in the headwaters to a low elevation of 925 feet near the mouth of the canyon at the Santa Clara River valley. The area surrounding the upper channel in Chiquito Canyon within the Newhall Ranch project area is primarily comprised of agricultural land (URS, 2003). In contrast to the vegetation found in the upper portion of Chiquito Canyon within the project area, the vegetation found in the downstream portion of the drainage within the project area is quite diverse, supporting scalebroom scrub, coast live oak woodlands, and Great Basin scrub.

The two square mile (1,295 acre) Long Canyon watershed is also tributary to the southern bank of the Santa Clara River. Approximately 845 acres of Long Canyon, or 65% of the watershed area, is located within the NRSP boundary, with the remainder being upstream in the Legacy Village subregion (see Figure 2-1). The drainage in the headwaters is aligned in a general west to east direction. The distance from the upper headwaters to the canyon mouth is approximately 18,350 lineal feet, with an average overall slope of 0.052 (PACE, 2006b). The major natural main stem drainage course within the watershed has an average slope in the lower reaches of the watershed of approximately 0.11. The topography for the watershed varies from a maximum elevation of 2,600 feet in the headwaters to a low elevation of 930 feet near the mouth of the canyon at the Santa Clara River valley. Both sides of this watershed contain habitat types comprised primarily of coastal sage scrub, with small pockets of chamise chaparral, and grassland present (URS, 2003). Within the stream channel, there is a mixture of grassland,

elderberry scrub, live oak woodland, alluvial scrub, great basin scrub, mixed chaparral, and alluvial scrub.

The 8.7 square mile (5,555.3 acre) Castaic Creek watershed is a tributary located north of the Santa Clara River. The total length of the mainstem channel is approximately 36,819 feet, with an average overall slope of 3.7 percent. The maximum elevation difference from the headwaters to the mouth of the creek at the Santa Clara River is 1,378 feet. Generally, the soils in the watershed are characterized as Saugus loam and are predominately classified as being in hydrologic soil group "B" (lower runoff potential). The associated vegetative cover within the watershed varies, but primarily consists of California coastal sage scrub.

The Adobe Canyon borrow site is located south of the Landmark Village tract map site and east of Long Canyon on the south side of the river. Adobe Canyon is characterized by sloping hillsides and adjacent agricultural use. The borrow site is dominated by coastal sage scrub, but also includes areas of coastal sage chaparral scrub, non-native grassland, and live oak woodland. Elevations on the borrow site range from approximately 920 feet (near the river) rising to 1,260 feet above mean sea level further south.

2.6 Existing Receiving Water Quality

Due to the size of the study area and the highly variable nature of wet weather surface water quality in the Santa Clara River throughout the study area, it was not appropriate to summarize water quality data for a single timeframe or location in order to establish baseline water quality conditions. As discussed above, flows in the Santa Clara River are highly episodic in nature and this characteristic can affect surface water quality considerably. The data summarized below, however, is recent and provides an accurate and reasonable characterization of existing water quality conditions that exist in the Project area. Data collected by the USGS at the Los Angeles County/Ventura County line also summarized below provides historical perspective of water quality within the Santa Clara River at the downstream Project boundary.

The existing wet and dry weather surface water quality in the Project area was characterized from available water quality monitoring data obtained from the following four sources (see Figure 2-1 for monitoring locations):

1. Newhall Ranch Tributary Stormwater Monitoring. Two storm events were monitored in Potrero Canyon, San Martinez Grande Canyon, Middle Canyon, Chiquito Canyon, and an unnamed tributary in Long Canyon. This data is relevant in terms of characterizing the existing stormwater runoff within the Project area. Although limited, this data is relevant in terms of characterizing the existing stormwater runoff within the Santa Clara River tributaries within the NRSP area as the conditions within these watersheds have not been altered since 2000.

2. Newhall WRP. The Newhall Ranch is required to conduct pre-startup water quality monitoring at upstream and downstream locations from the outfall of the approved Newhall WRP. Wet and dry weather monitoring data were collected from two stations in the SCR from the spring of 2004 through 2007: one station is near the downstream boundary of the NRSP area near to the proposed WRP outfall location, and the second is about 2.5 miles further downstream. Additionally, dry weather monitoring has been conducted at three stations (RSW-001U, RSW-001D, RSW-002D) as required by the Newhall WRP NPDES Permit (LARWQCB, 2007). These stations are referred to below as the "Newhall WRP NPDES Stations."

3. LA County Monitoring. The County of Los Angeles conducts in-stream monitoring on the mainstem of the SCR at a mass emission station located at The Old Road, upstream of the Project area. Both dry weather and wet weather monitoring data are available. Wet weather monitoring data are available from November 2002 through February 2009. The LA County monitoring data are the most current and are the only source of wet weather monitoring in the SCR immediately upstream of the Project area.

4. USGS Monitoring. The USGS collected a large number of water quality data in the SCR near the County line from 1951 through 1995. These data provide a historical perspective of wet and dry weather water quality in the SCR immediately downstream from the Project area.

2.6.1 Wet Weather Water Quality Monitoring

Wet Weather Monitoring Locations and Rainfall Conditions

NRSP Area Stormwater Monitoring. Newhall Land conducted stormwater monitoring of tributary streams in the NRSP area to characterize the existing surface water quality during wet weather conditions (the monitoring data is provided in Appendix C). Stormwater samples were collected during two storm events in March 2001 at five monitoring locations (Stations A-E) shown on Figure 2-1. Three of the five monitoring stations were located at the mouths of SCR tributaries in Potrero Canyon (Sta. A), San Martinez Grande Canyon (Sta. B), and Middle Canyon (Sta. D). The other two monitoring stations were located on tributaries upstream from the mainstem of the SCR; one was just downstream of the community of Val Verde in Chiquito Canyon (Sta. E) and one was on an unnamed tributary in Long Canyon, 0.25 mile upstream of the 'Onion Field' (Sta. C). Aside from Station E, which is downgradient of existing residential development, the land uses in the areas tributary to the Stations A, B, C, and D are predominately open space with some agriculture and oil and gas operations.

Table 2-3 lists the rainfall depth and duration of the two monitored storm events. The first storm was a small event (0.2 inches) that was likely just large enough to result in stormwater runoff. The depth of the second event was larger than the median storm depth (0.6 inches) at the nearby National Climatic Data Center (NCDC) Newhall rain gauge (see location on Figure 2-1). The

median depth of 0.6 inches is based on a storm event analysis which identified 613 storms exceeding 0.1 inches that occurred from October 1968 to December 2008. The average storm duration for storms greater than 0.1" in the 40 year Newhall rain gauge record is 11.4 hours.

Table 2-3: Depth and Duration of Storms Monitored at Project Site

Date	Depth (in)[1]	Duration (hours)[1]
03/06/01	0.2	3
03/08/01	0.7	10

[1] Based on rainfall measured at the Newhall rain gauge.

Newhall WRP Pre-Startup Monitoring. Newhall Land has conducted pre-startup receiving water quality monitoring for the approved Newhall Ranch WRP at two locations in the SCR (see Figure 2-1):

- NR1 is located in the SCR 300 feet upstream of the WRP outfall location, and
- NR3 is located in the SCR approximately 7,500 feet downstream of the WRP outfall.

Five storms with rainfall depths ranging from 0.1 to 0.6 inch were sampled at NR1 and NR3 and one very large storm with a depth of 4.45 inches was sampled at NR3 (Table 2-4). Grab sampling methods were used. Table 2-4 shows the depths and durations of storm events monitored.

Table 2-4: Depth and Duration of Storms Monitored for Newhall WRP

Date	Depth (in)[1]	Duration
12/07/04	0.12	6
2/17/05[3]	0.6	12
2/18/05[2,3]	4.45	12
11/9/05	0.12	6
11/10/05[3]	0.2	1
2/17/06	0.31	7

[1] Based on rainfall measured at the Newhall rain gauge.
[2] NR-3 only sampled
[3] Estimated due to lack of gauge data.

LA County Department of Public Works Monitoring Data. The Los Angeles County Department of Public Works (LACDPW) has conducted mass emission dry and wet weather monitoring in the Santa Clara River for seven seasons - from 2002 through 2009 (LACDPW, 2009). The monitoring station (S29) is located in the Santa Clara River at The Old Road (Figure 2-1). It is approximately two miles upstream from the eastern boundary of the NRSP area. The monitoring station is downstream of the Saugus WRP and the City of Santa Clarita and upstream of the Valencia WRP. The monitoring station is intended to provide long-term information about

water quality trends in areas with heterogeneous land uses and has a tributary area of 411 square miles.

Monitoring at the mass emission station included twenty-six storm events over 7 years. Composite samples were collected for most parameters; grab sampling was used for bacteria analyses. Table 2-5 lists the rainfall depths and durations of the twenty-six monitored storm events based on hourly rainfall measurements at the Newhall rain gauge. The storm event on 1/14/2006 was not included in the data summaries as it was less than 0.1", the minimum storm depth to generate runoff. The median of the twenty-five remaining storm events is 0.64", roughly equivalent to the median storm depth for the Newhall rain gauge from 1968 through 2008 (0.60 inches).

Table 2-5: Depth and Duration of Storms Monitored by LACDPW at S29

Date	Rainfall Depth (inches)[1]	Storm Duration (hours)[1]
11/8/2002	1.6	21
12/16/2002	1.9	5
2/11/03	8.0	32
3/15/03	2.0	16
10/31/2003	0.30	4
12/25/2003	1.80	14
1/1/2004	0.4	9
10/17/2004	0.64	7
10/26/2004	2.22	13
1/7/2005	9.99	92
10/17/2005	1.61	14
12/31/2005	0.6	4
1/14/2006[2]	0.08	2
2/17/2006	0.32	7
12/9/2006	0.47	2
12/16/2006	0.12	2
1/30/2007	0.44	16
2/19/2007	0.24	5
2/22/2007	0.32	3
9/21/2007	0.98	16
11/29/2007	0.34	6
12/6/2007[3]	0.43	48
11/26/2008	1.22	17

Date	Rainfall Depth (inches)[1]	Storm Duration (hours)[1]
12/15/2008	1.22	19
2/5/2009[4]	2.2	40
2/13/2009[4]	0.32	3

[1] Based on rainfall measured at the Newhall rain gauge

[2] This storm was not included in monitoring summary since it is <0.1"

[3] The Newhall gauge noted accumulations for this storm event. LA County recorded this storm with a depth of 0.43" and duration of 9 hours.

[4] The depths and durations for storms in 2009 are those recorded by LA County.

USGS Water Quality Monitoring Data. The US Geological Survey (USGS) has collected stream flow and water quality data at a number of locations in the SCR watershed (http://waterdata.usgs.gov/nwis). Among the largest data sets are flow and water quality data collected at USGS station 11108500 located on the Santa Clara River just downstream of the Los Angeles / Ventura County Line. This station is located approximately one mile downstream of the NRSP area (Figure 2-1), and downstream of both existing WRPs.

The USGS collected water quality data between April 1951 and October 1995. These data thus provide a historical perspective of water quality in the SCR within the NRSP area.

Data presentation. To facilitate interpretation, the wet weather water quality data were grouped into two categories depending on the depth of 2-day antecedent rainfall measured at the Newhall rain gauge:

1. ***0.1 – 1 inches.*** Rainfall depths that would likely produce runoff volumes characteristic of more frequent, smaller storm events.

2. ***> 1 inch***. Rainfall depths that would likely produce runoff volumes characteristic of larger, less frequent storm events.

Selected General Constituents

The selected general constituents examined were Total Suspended Solids (TSS), Total Dissolved Solids (TDS), Hardness and Chloride (see Section 4 for a discussion of pollutant selection). TSS is a measure of the particulate matter suspended in water. Total dissolved solids (TDS) are a measure of the dissolved cations and anions, primarily inorganic salts (calcium, magnesium, potassium, sodium, chlorides and sulfates). TDS is an impairing pollutant in Reach 3 of the SCR as listed in the State's 2006 303(d) list of impaired water bodies. High TDS levels can impair agricultural, municipal supply, and groundwater recharge beneficial uses.

Hardness and chloride are important components of TDS. Hardness is a measure of the polyvalent cations, primarily calcium and magnesium. It is expressed as an equivalent concentration of calcium carbonate ($CaCO_3$). Hardness measurements are important because the toxicity of metals (and the associated water quality objectives) is an inverse function of the

hardness. Chloride comprises a large proportion of the TDS and is responsible for impairments in its own right. High levels of chloride in Santa Clara River Reaches 3, 5, and 6 are causing impairment of listed beneficial uses for agricultural irrigation. Irrigation of salt sensitive crops, such as avocados and strawberries, with water containing elevated levels of chloride can result in reduced crop yields. A chloride TMDL was approved for these reaches in 2005.

Results for concentrations of TSS, TDS, chloride and hardness for the four datasets are listed in Table 2-6 through Table 2-9. Rather than measuring TDS, the USGS station has recorded specific conductance (that is, the extent to which the sample conducts an electric current), which is related to TDS concentration. TDS concentration can be estimated as 0.55 to 0.9 times the specific conductance (Sawyer et al, 1994).

Table 2-6: Average Concentrations of General Constituents and Nutrients from Newhall Ranch Tributary Stormwater Monitoring, March 2001

Constituent	Site A Mouth of Potrero	Site B Mouth of San Martinez Grande	Site C Long Canyon Upstream of Onion Field	Site D Mouth of Middle Canyon	Site E Middle of Chiquito
TSS (mg/L)	835	41,100	36,000	5,650	6,645
TDS (mg/L)	7,380	2,825	190	160	205
Hardness (mg/L as CaCO3)	2,225	1,205	147	59	107
Chloride (mg/L)	870	125	3	3	11

Table 2-7: Newhall Ranch WRP Startup Wet Weather Water Quality Data for General Constituents and Nutrients in the Santa Clara River, 2004 - 2006

Constituent	2-day Antecedent Rainfall (inches)	Sample Site	Minimum (mg/L)	Maximum (mg/L)	Average (mg/L)
TSS	0.1 – < 1.0	NR1	32	107	58
		NR3	32	235	112
	≥ 1.0	NR3	-	-	43,360
TDS	0.1 – < 1.0	NR1	622	1,136	855
		NR3	698	2,020	1,076
	≥ 1.0	NR3	-	-	2,100
Hardness (mg/L as $CaCO_3$)	0.1 – < 1.0	NR1	304	464	387
		NR3	352	670	475
	≥ 1.0	NR3	-	-	832

- = no or insufficient data; NA – not applicable

Table 2-8: LACDPW Stormwater Monitoring for Selected General Constituents at the SCR Mass Emission Station (S29) during 2002-2009

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	Minimum	Maximum	Average
TSS (mg/L)	0.1 – < 1.0	14	26	2,202	729
	≥ 1.0	11	53	6,591	1,482
TDS (mg/L)	0.1 – < 1.0	14	130	732	419
	≥ 1.0	11	28	364	197
Hardness (mg/L)	0.1 – < 1.0	14	70	428	223
	≥ 1.0	11	15	170	101
Chloride (mg/L)	0.1 – < 1.0	14	16	118	60
	≥ 1.0	11	3	52	22

- = no or insufficient data; NA – not applicable

Table 2-9: USGS Water Quality Data for Selected General Constituents in the Santa Clara River at the County Line during 1951 – 1995

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
TSS (mg/L)	0.1 – < 1.0	10	10	248	4,730	2,291
	≥ 1.0	41	41	107	51,200	10,711
Specific Conductance (uS/cm)	0.1 – < 1.0	33	33	831	4,220	2,246
	≥ 1.0	42	42	637	3,240	1,309
Hardness (mg/L)	0.1 – < 1.0	27	27	270	1,500	773
	≥ 1.0	37	37	250	1,200	546
Chloride (mg/L)	0.1 – < 1.0	34	34	21	290	122
	≥ 1.0	39	39	14	192	61

TSS. It is generally expected that TSS concentrations in alluvial streams can be greatly elevated during storm runoff because of the combination of high sediment supply and a high capacity for instream transport and erosion. TSS concentrations in Table 2-6 to Table 2-9 are sometimes very high, due to the highly erodible, easily transportable, sandy alluvial soils and sediments. Highest TSS concentrations were measured at some of the tributary canyons (Table 2-6), but were also observed in the SCR (Table 2-8 and Table 2-9). These latter results show the capacity of high flows in the Santa Clara River for sediment transport and support the conclusion that large rainfall events result in a "reset" of the main channel. As concluded by Balance Hydrologics (2005), concepts of "normal" or "average" sediment-supply and flow conditions have limited value in this "flashy" environment, where episodic storm and wildfire events have enormous

influence on sediment and storm flow conditions. In the Santa Clara River, a large portion of sediment movement events can occur in a matter of hours or days.

Average and maximum concentrations are much higher in the larger storms than the smaller storms at the downstream sites on the SCR. This pattern is also evident in the upstream LACDPW station, to a lesser extent.

The water quality objective for TSS in the Basin Plan is a narrative standard, which states, "water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses".

TDS. Stormwater monitoring data collected in the NRSP tributaries (Table 2-6) show greatly differing TDS levels among the five monitoring stations. Measured TDS concentrations were very high at Sites A and B, while TDS concentrations at the other three sites were low. It is highly unlikely that this is a land use effect. Elevated TDS levels in runoff at Site A and B is likely a result of the natural soil properties of the marine layers of the Pico Formation, and the high groundwater table conditions in these two canyons, suggesting that groundwater discharges to the streams contributed to the elevated TDS levels. These greatly differing dissolved solid (TDS) concentrations are also reflected in some of the components that make up the TDS (chloride and hardness) as described below.

Average concentrations of TDS in the Santa Clara River were moderate to high, ranging from 216 mg/L to 2,100 mg/L. Using an estimate of 0.64 times the specific conductance for the USGS data, the TDS concentrations at this station averaged around 1,400 mg/L for storm flows. The Basin Plan objective for TDS in Santa Clara River Reach 5 is 1,000 mg/L.

Much higher average concentrations were observed at the three downstream SCR stations (NR-1, NR-3, USGS) compared with the upstream LACDPW station, and this could be due to their location downstream of the tributaries represented by Sites A and B, with their much higher salt content.

TDS concentrations were generally lower in the larger storms, reflecting a dilution effect.

Hardness. Hardness is a measure of the multivalent cations in water, principally calcium, magnesium, strontium, iron, and manganese (Sawyer et al, 1994). These cations are capable of reacting with soap to form precipitates and with certain anions to form scale. The hardness in water is derived largely from contact with soil and rock formations, and affects the CTR values for certain metals as discussed above. Waters with a hardness concentration from 150 mg/L to 300 mg/L as $CaCO_3$ are considered hard; waters with a hardness concentration above 300 mg/L as $CaCO_3$ are considered very hard.

The stormwater monitoring data for hardness were analogous to the data for TDS. Hardness concentrations were very high at the tributary Sites A and B, and low to moderate at the other three tributary sites. High hardness at Sites A and B could be due to natural high levels of calcium and magnesium in the local soils and sedimentary formations (such as lime and gypsum deposits), and the high groundwater table conditions in these two canyons, suggesting again that groundwater discharges contributed to the elevated hardness levels.

In the SCR, average hardness values were greater downstream (NR3, NR1, USGS sites – Table 2-7, Table 2-9) than at the LACDPW station (Table 2-8). This is most likely due to the influence of tributary inflows of high hardness waters (such as measured at Sites A and B – Table 2-6), other groundwater inputs, and agricultural return flows that enter the Santa Clara River between these stations. However, the magnitude of hardness concentrations was somewhat inconsistent, with the USGS station (Table 2-9) showing higher average hardness concentrations than those measured at NR-1 and NR-3 (Table 2-7).

The average hardness concentration decreased with larger antecedent rainfall depth, as was found for TDS concentrations.

Chloride. Similar to TDS and hardness, monitoring data collected in the NRSP tributaries (Table 2-6) found very high chloride concentrations at Site A, high levels at Site B, and low concentrations at the remaining three sites.

As with the other dissolved ionic parameters (TDS and hardness) the average chloride concentrations at the LACDPW station (
Table 2-8) were lower than those measured at the downstream USGS site (Table 2-9).

Overall, the average chloride concentrations during recent stormwater monitoring were highly variable and ranged between 3 mg/L and 120 mg/L, with the exception of the very high chloride concentrations detected at the mouth of Potrero Canyon (Site A). Average chloride concentration at the USGS station was about 61 mg/L for storm flows. The Basin Plan objective for chloride is 100 mg/L.

Nutrients

The major nutrients nitrogen and phosphorus are described here. Phosphorus was measured as total phosphorus (TP) and sometimes as dissolved phosphorus. Dissolved phosphorus is the more bioavailable form of phosphorus compared to TP, which is often made up of a high proportion of particulate phosphorus. Nitrogen is measured variously as nitrate, nitrite, ammonia, and total Kjeldahl nitrogen (TKN). TKN is the measure of ammonia plus the organic forms of nitrogen. Nitrate, nitrite, and ammonia are the more bioavailable forms of nitrogen, and of these, nitrate (or nitrate + nitrite) has the higher concentration in natural waters and is more important than ammonia as a nutrient. Table 2-10 through Table 2-13 summarizes available data for these

nutrients. Only nitrate+nitrite (N) was measured in the Newhall Ranch Tributary Stormwater Monitoring.

Table 2-10: Average Concentrations of Nitrate from Newhall Ranch Tributary Stormwater Monitoring in March 2001

Constituent	Site A Mouth of Potrero	Site B Mouth of San Martinez Grande	Site C Long Canyon Upstream of Onion Field	Site D Mouth of Middle Canyon	Site E Middle of Chiquito
Nitrate + Nitrite-N (mg/L)	17.5	3.0	1.6	15.3	2.8

Table 2-11: Newhall WRP Pre-Startup Wet Weather Water Quality Data for Selected Nutrients in the Santa Clara River during 2004 - 2006

Constituent	2-day Antecedent Rainfall (inches)	Sample Site	No. of Samples	No. of Detects	Minimum (mg/L)	Maximum (mg/L)	Average (mg/L)
Total Phosphorus	0.1 – < 1.0	NR1	5	5	0.4	0.5	0.4
		NR3	5	5	0.3	0.7	0.4
	≥ 1.0	NR3	1	1	13.4	13.4	13.4
Nitrate as N	0.1 – < 1.0	NR1	5	5	1.9	4.8	3.2
		NR3	5	5	2.3	3.7	3.0
	≥ 1.0	NR3	1	1	1.4	1.4	1.4
Nitrite as N	0.1 – < 1.0	NR1	5	0	<0.005	<0.005	<0.005
		NR3	5	0	<0.005	<0.005	<0.005
	≥ 1.0	NR3	1	0	<0.005	<0.005	<0.005
Ammonia as N	0.1 – < 1.0	NR1	5	4	<0.005	0.3	0.2
		NR3	5	5	0.02	0.1	0.1
	≥ 1.0	NR3	1	1	0.5	0.5	0.5
TKN as N	0.1 – < 1.0	NR1	5	4	<0.04	0.7	0.3
		NR3	5	4	<0.04	0.6	0.4
	≥ 1.0	NR3	1	1	46.0	46.0	46.0

- = no or insufficient data; NA – not applicable

Table 2-12: LACDPW Stormwater Monitoring of Nutrients at the SCR Mass Emission Station (S29) during 2002-2005

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved phosphorus (mg/L)	0.1 – < 1.0	14	14	0.05	0.43	0.23
	≥ 1.0	11	11	0.10	0.45	0.26
Total phosphorus (mg/L)	0.1 – < 1.0	14	14	0.30	1.29	0.62
	≥ 1.0	11	11	0.18	0.94	0.54

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
Nitrate-N (mg/L)	0.1 – < 1.0	14	9	<0.08	1.85	0.81
	≥ 1.0	11	9	<0.08	1.36	0.74
Nitrite-N (mg/L)	0.1 – < 1.0	14	4	<0.03	1.00	0.12
	≥ 1.0	11	3	<0.03	0.87	0.13
Ammonia-N (mg/L)	0.1 – < 1.0	14	6	<0.03	1.35	0.17
	≥ 1.0	11	8	<0.03	1.09	0.23
TKN as N (mg/L)	0.1 – < 1.0	14	14	0.70	8.70	2.61
	≥ 1.0	11	11	0.45	31.7	4.32

- = no or insufficient data; NA – not applicable

Table 2-13: USGS Water Quality Data for Selected Nutrients in the Santa Clara River at the County Line during 1951 to 1995

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved phosphorus (mg/L)	0.1 – < 1.0	3	3	0.35	0.66	0.46
	≥ 1.0	1	1	0.01	0.01	0.01
Total phosphorus (mg/L)	0.1 – < 1.0	5	5	0.81	1.8	1.28
	≥ 1.0	2	2	0.63	1.4	1.02
Ammonia as N (mg/L)	0.1 – < 1.0	3	3	0.03	0.39	0.16
	≥ 1.0	0	0	-	-	-
Nitrate + Nitrite as N (mg/L)	0.1 – < 1.0	7	7	0.87	4	2.1
	≥ 1.0	4	4	1.2	2	1.7
TKN as N (mg/L)	0.1 – < 1.0	1	1	0.64	0.64	0.64
	≥ 1.0	1	1	0.69	0.69	0.69
Total Nitrogen (mg/L)	0.1 – < 1.0	2	2	0.6	2.2	1.4
	≥ 1.0	2	2	3.5	4.4	4.0

- = no or insufficient data

Phosphorus. Recent wet weather monitoring showed somewhat consistent total phosphorus levels, averaging about 0.4 to 0.6 mg/L. An exception was the large storm sample (>1.0 inch) collected at station NR-3, which measured 13.4 mg/L. This was likely due the high concentration of total suspended solids measured during the same storm event, because total phosphorus is predominately found in the particulate-phase in stormwater runoff. Historical average total phosphorus concentrations at the USGS station were somewhat higher than recent results at 1.0 to 1.3 mg/L and appeared to be somewhat independent of storm event size. The Basin Plan water quality objective for phosphorus is a narrative standard, which states, “waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance of adversely affects beneficial uses”.

Nitrogen. Nitrate + nitrite-nitrogen was the only nutrient measured in the NRSP tributary stormwater monitoring. As shown in Table 2-10, measured nitrate-nitrogen concentrations were generally low (less than 3 mg/L as N) at three of the sites, and were elevated at Sites A and D (17.5 mg/L and 15.3 mg/L, respectively). High nitrate levels can be associated with runoff from agricultural areas and nurseries, or associated with excessive fertilization of landscaping in residential areas; however, Station E, which is downstream of residential development, showed relatively low nitrate concentrations.

Most of the more recent nitrate monitoring data summarized in Table 2-10, Table 2-11, and Table 2-12 were relatively low (averaging 0.8 to 3.0 mg/L). The average historical nitrate-N + nitrite-N concentrations at the USGS station were roughly similar, varying from 2.1 mg/L for lower storm flows to 1.7 mg/L for higher storm flows.

Average ammonia concentrations were low and ranged from 0.1 to 0.5 mg/L. The ammonia water quality objectives in the Santa Clara River Nitrogen Compounds TMDL range from 3.4 mg/L to 5.5 mg/L (one hour average) and 1.2 mg/L to 2.0 mg/L (30-day average).

Average total Kjeldahl nitrogen (TKN) concentrations generally ranged between the concentrations found for ammonia and nitrate (about 0.4 to 4.3 mg/L). One exception was the concentration found in the large storm at NR-3, which measured 46 mg/L. As with total phosphorus, the organic forms of nitrogen in stormwater runoff are generally in the particulate-phase, and this result correlated with the high levels of total phosphorus and suspended solids measured during this same event as described above.

Selected Metals and Pesticides

The metals cadmium (Cd), copper (Cu), lead (Pb), and zinc (Zn) can be toxic at high concentrations. They occur naturally in soils and sediments, and can be present in urban runoff. Aluminum is one of the more abundant elements in the earth's crust. The organophosphorous pesticides chlorpyrifos and diazinon are especially toxic to a number of aquatic organisms and in the past have been frequently detected downstream from urban and agricultural land uses. Cyanide is a highly toxic substance and originates from both man-made and natural sources.

Table 2-14 though Table 2-17 summarize the data for these metals and pesticides in the tributaries and the Santa Clara River. Cyanide was only measured at the LACDPW Mass Emission station. Available data for metals at the USGS station were very limited. For copper and lead, there were a considerable number of non-detects with very high detection limits. Therefore, comparison of the USGS data for copper, lead, and zinc with the recent monitoring information is considered inappropriate. Metals data were not collected in the one large storm event sampled for the Newhall Ranch WRP pre-startup monitoring; thus summarized data for this station represent storms less than one inch in depth.

Table 2-14: Average Concentration of Metals from Newhall Ranch Tributary Stormwater Monitoring, March 2001

Constituent	Site A Mouth of Potrero	Site B Mouth of San Martinez Grande	Site C Long Canyon Upstream of Onion Field	Site D Mouth of Middle Canyon	Site E Middle of Chiquito Canyon
Total Copper (µg/L)	15	175	170	10	70
Total Lead (µg/L)	6.1	54	95	8	37
Total Zinc (µg/L)	40	330	330	30	225
Total Cadmium (µg/L)	0.3	11.2	2.0	0.4	1.9

Table 2-15: Newhall Ranch WRP Pre-Startup Wet Weather Water Quality Data for Metals and Pesticides in the Santa Clara River, 2004 - 2006

Constituent	2-day Antecedent Rainfall (inches)	Sample Site	No. of Samples	No. of Detects	Minimum(µg/L)	Maximum (µg/L)	Average (µg/L)
Dissolved Aluminum (µg/L)	0.1 – < 1.0	NR1	1	1	27	27	27
		NR3	1	1	19	19	19
Total Aluminum (µg/L)	0.1 – < 1.0	NR1	1	1	740	740	740
		NR3	1	1	770	770	770
Dissolved Copper (µg/L)	0.1 – < 1.0	NR1	1	1	4.6	4.6	4.6
		NR3	1	1	3.6	3.6	3.6
Total Copper (µg/L)	0.1 – < 1.0	NR1	2	2	4.6	5.2	4.9
		NR3	2	2	4.8	7.0	5.9
Dissolved Lead (µg/L)	0.1 – < 1.0	NR1	1	0	<0.07	<0.07	-
		NR3	1	0	<0.07	<0.07	-
Total Lead (µg/L)	0.1 – < 1.0	NR1	2	2	0.6	1.3	1.0
		NR3	2	2	0.6	0.9	0.8
Dissolved Zinc (µg/L)	0.1 – < 1.0	NR1	1	1	12	12	12
		NR3	1	1	8.7	8.7	8.7
Total Zinc (µg/L)	0.1 – < 1.0	NR1	2	2	13	22	18
		NR3	2	2	12	18	15
Diazinon	0.1 – < 1.0	NR1	1	0	<0.01	<0.01	<0.01
		NR3	1	0	<0.01	<0.01	<0.01
Chlorpyrifos	0.1 – < 1.0	NR1	1	0	<0.6	<0.6	<0.6
		NR3	1	0	<0.6	<0.6	<0.6

- = no or insufficient data; NA – not applicable

[1] Water Quality Standards are CTR acute criteria calculated with minimum measured hardness value for monitoring location.

Table 2-16: LACDPW Stormwater Monitoring for Metals, Pesticides, and Cyanide at the SCR Mass Emission Station (S29) during 2002-2005

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved aluminum (µg/L)	0.1 – < 1.0	14	4	<50	1,390	264
	≥ 1.0	11	4	<50	3,680	420
Total aluminum (µg/L)	0.1 – < 1.0	14	14	383	18,000	5,770
	≥ 1.0	11	11	131	19,650	5,161
Dissolved copper (µg/L)	0.1 – < 1.0	14	14	3.3	11.5	6.4
	≥ 1.0	11	11	3.8	22.6	8.4
Total copper (µg/L)	0.1 – < 1.0	14	14	7.3	91.3	29.8
	≥ 1.0	11	11	9.4	53.3	31.1
Dissolved lead (µg/L)	0.1 – < 1.0	14	3	<0.44	3.3	0.5
	≥ 1.0	11	8	<0.44	12.5	2.4
Total lead (µg/L)	0.1 – < 1.0	14	14	1.4	39	8.6
	≥ 1.0	11	11	1.1	110	29.9
Dissolved zinc (µg/L)	0.1 – < 1.0	14	12	<1	27	14
	≥ 1.0	11	11	12	37	24
Total zinc (µg/L)	0.1 – < 1.0	14	14	11	292	71
	≥ 1.0	11	11	42	353	126
Dissolved cadmium (µg/L)	0.1 – < 1.0	14	0	<0.05	<0.05	<0.05
	≥ 1.0	11	1	<0.05	0.74	0.10
Total cadmium (µg/L)	0.1 – < 1.0	14	8	<0.05	3.47	0.53
	≥ 1.0	11	8	<0.05	1.30	0.75
Chlorpyrifos (µg/L)	0.1 – < 1.0	14	0	<0.02	<0.02	<0.02
	≥ 1.0	11	0	<0.02	<0.02	<0.02
Diazinon (µg/L)	0.1 – < 1.0	14	3	<0.003	0.41	0.04
	≥ 1.0	11	5	<0.003	0.43	0.07
Cyanide (µg/L)	0.1 – < 1.0	14	4	<0.005	0.01	0.003
	≥ 1.0	11	3	<0.005	0.59	0.06

Table 2-17: USGS Water Quality Data for Selected Metals, Pesticides and Indicator Bacteria in the Santa Clara River at the County Line during 1951 to 1995

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved Copper (µg/L)	0.1 – < 1.0	4	0	-	-	-
	≥ 1.0	0	0	-	-	-
Total Copper (µg/L)	0.1 – < 1.0	1	1	30	30	30
	≥ 1.0	0	0	-	-	-
Dissolved Lead (µg/L)	0.1 – < 1.0	39	4	1	23	7.8
	≥ 1.0	4	0	-	-	-
Total Lead (µg/L)	0.1 – < 1.0	3	0	-	-	-
	≥ 1.0	1	0	-	-	-
Dissolved Zinc (µg/L)	0.1 – < 1.0	4	1	-	10	10
	≥ 1.0	0	0	-	-	-
Total Zinc (µg/L)	0.1 – < 1.0	1	1	150	150	150
	≥ 1.0	0	0	-	-	-
Diazinon (µg/L)	0.1 – < 1.0	1	1	0.02	0.02	0.02
	≥ 1.0	0	0	-	-	-

- = no or insufficient data

Metals. Table 2-14 presents average total copper, lead, zinc, and cadmium concentrations measured in the NRSP tributary stormwater monitoring. Total copper, lead, and zinc measured at tributary Sites B and C were much higher than the concentrations measured at Sites A and D. Concentrations at Site E fell in the middle of the measured range. Elevated total metal concentrations are often associated with elevated TSS levels, although this trend is not evident in the tributary monitoring data. The average total copper concentrations at Sites B, C, and E were greater than the CTR acute copper criterion. The average total copper concentrations ranged from 10 µg/L to 175 µg/L; the CTR acute total copper criterion for a hardness concentration of greater than 400 mg/L is 52 µg/L. The average total lead and total zinc concentrations in all the tributaries were below the CTR acute criteria. The average total lead concentrations ranged from 6.1 µg/L to 95 µg/L; the CTR acute total lead criterion for a hardness concentration of greater than 400 mg/L is 480 µg/L. The average total zinc concentrations ranged from 30 µg/L to 330 µg/L; the CTR acute total zinc criterion for a hardness concentration of greater than 400 mg/L is 390 µg/L.

Concentrations of dissolved copper (3.6 to 8.4 µg/L) were below the CTR acute criteria for the average hardness of 250 mg/L (32 µg/L). Concentrations of total copper measured in the Santa Clara River (4.6 to 91 µg/L, total copper) exceeded the respective CTR acute criteria for the average hardness of 250 mg/L (33 µg/L, total copper) in 9 of 25 samples for the LACDPW station (there were no exceedances at other SCR stations). Concentrations of dissolved and total

lead measured in the Santa Clara River (<0.07 µg/L to 23 µg/L, dissolved lead; 0.8 to 110 µg/L, total lead) were well below the respective CTR acute criteria for the average hardness of 250 mg/L (170 µg/L, dissolved lead; 260 µg/L, total lead). Concentrations of dissolved zinc measured in the Santa Clara River (8.7 µg/L to 37 µg/L, dissolved zinc) were well below the respective CTR acute criteria for the average hardness of 250 mg/L (250 µg/L, dissolved zinc); concentrations of total zinc in the Santa Clara River range from 11 to 353 µg/L, with data from the LACDPW gauge exceeding the CTR criteria for an average hardness of 250 mg/L (260 µg/L, total zinc) in 2 of 25 samples (there were no exceedances at the other SCR stations).

Measured aluminum concentrations showed a very wide range of concentrations at the mass emission station (Table 2-16).

Pesticides. Data for pesticides are very limited. Chlorpyrifos was not detected at LACDPW station, and Diazinon was detected in about a third of samples (8/25) with an average concentration of 0.04 µg/L in small storms and 0.07 µg/L in larger storms (Table 2-16). Diazinon and Chlorpyrifos were not detected further downstream in the SCR during Newhall WRP wet weather sampling (Table 2-15) but were detected in the one wet weather sample taken in the historical USGS data (Table 2-17). The CTR acute criterion for diazinon is 0.17 µg/L. The diazinon criterion derived by the CDFG is 0.08 µg/L (Marshack, 2003).

Cyanide. Cyanide was detected in 7 of 25 samples collected at the LACDPW station. Concentrations observed at the LACDPW station were very low, exceeding the CTR criterion for freshwater acute aquatic of 22 µg/L in only one instance (Table 2-16).

Fecal Indicator Bacteria

Pathogens such as viruses, bacteria, and protozoa that cause illness in humans are difficult to measure. Fecal indicator bacteria (FIB) such as Total Coliform, Fecal Coliform and Enterococci are commonly measured instead, and their presence indicates the presence of fecal contamination and the potential presence of associated pathogenic organisms. However, it does not indicate the source of the contamination and there are numerous natural and anthropogenic sources of pathogen indicators. Table 2-18 through 2-21 summarize FIB data for the four datasets. Averages are presented as geometric means.

Table 2-18: Concentrations for Fecal Indicator Bacteria from Newhall Ranch Tributary Stormwater Monitoring, 2001

Constituent	Site A Mouth of Potrero	Site B Mouth of San Martinez Grande	Site C Long Canyon Upstream of Onion Field	Site D Mouth of Middle Canyon	Site E Middle of Chiquito
Total coliform (MPN/100ml)	38,700	>160,000	120,000	>89,400	>19,600
Fecal coliform (MPN/100ml)	3,300	590	4,200	>19,600	19,600

NA – not applicable

Table 2-19: Newhall Ranch WRP Startup Wet Weather Water Quality Data for Fecal Indicator Bacteria in the Santa Clara River, 2004 - 2006

Constituent	2-day Antecedent Rainfall (inches)	Sample Site	No. of Samples	No. of Detects	Minimum	Maximum	Geometric Mean
Fecal coliform (MPN/100mL)	0.1 – < 1.0	NR1	5	4	<1	900	87
		NR3	5	4	<1	5,000	258
	≥ 1.0	NR3	1	1	≥1,600	≥1,600	≥1,600
Total coliform (MPN/100mL)	0.1 – < 1.0	NR1	5	4	<1	1,600	284
		NR3	5	4	<1	13,000	549
	≥ 1.0	NR3	1	1	≥1,600	≥1,600	≥1,600

- = no or insufficient data; NA – not applicable

Table 2-20: LACDPW Stormwater Monitoring for Fecal Indicator Bacteria at the SCR Mass Emission Station during 2002-2005

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Geometric Mean
Total coliform (MPN/100mL)	0.1 – < 1.0	14	14	17,000	1,600,000	101,000
	≥ 1.0	11	11	50,000	500,000	198,000
Fecal coliform (MPN/100mL)	0.1 – < 1.0	14	14	230	300,000	7,000
	≥ 1.0	11	11	3,000	300,000	36,000
Fecal Enterococci (MPN/100mL)	0.1 – < 1.0	14	14	800	300,000	27,000
	≥ 1.0	11	11	9,000	500,000	68,000

[1] Represents the geomean of the data.

Table 2-21: USGS Water Quality Data for Fecal Indicator Bacteria in the Santa Clara River at the County Line during 1951 - 1995

Constituent	2-day Preceding Rainfall (inches)	No. of Samples	No. of Detects	Minimum	Maximum	Geo-mean
Fecal Coliform (CFU/100mL)	0.1 – < 1.0	3	3	80	720	300
	≥ 1.0	1	1	-	-	2,700

- = no or insufficient data

Concentrations of total and fecal coliform bacteria in wet weather flows at all tributary monitoring stations, the Newhall Ranch WRP stations, and the County's mass emission station were highly variable and sometimes very high, consistent with other stormwater data throughout the region. Fecal coliform concentrations ranged from <1 Most Probable Number (MPN) per 100 milliliters (MPN/100 mL) to 300,000 MPN/100 mL. Average bacteria concentrations at the lower stations were significantly lower, but still elevated, more so during larger storms. In waters designated for water contact recreation (REC-1), the Basin Plan objective for fecal coliform is a log mean of 200/100 mL (based on a minimum of not less than 10 percent of total samples

during any 30-day period), nor shall more 10 percent of the total number of samples during any 30-day period exceed 400/100 mL.

Summary

Table 2-22 and Table 2-23 summarize the average values from wet weather monitoring data for all monitoring locations within the Project area.

Table 2-22: Summary of Average Wet Weather Monitoring Data (2-Day Antecedent Rainfall of 0.1 - 1.0 in)

	LACDPW Mass Emission Station	**NRSP Area Tributary Monitoring**					**Newhall WRP Startup Monitoring**		**USGS Wet Weather Monitoring**
Constituent	***S29***	***Site A***	***Site B***	***Site C***	***Site D***	***Site E***	***NR1***	***NR3***	***USGS***
General and Conventional Parameters									
TSS (mg/L)	729	835	41,100	36,000	5,650	6,645	58	112	2,291
TDS (mg/L)	419	7,380	2,825	190	160	205	855	1,076	1,437 [1]
Hardness (mg/L)	223	2,225	1,205	147	59	107	387	475	773
Chloride (mg/L)	59.6	870	125	3	3	11	-	-	122
Nutrients									
Total P (mg/L)	0.62	-	-	-	-	-	0.4	0.4	1.28
Nitrate-N (mg/L)	0.81	17.5[2]	3.0[2]	1.6[2]	15.3[2]	2.8[2]	3.2	3.0	2.1[2]
Nitrite-N (mg/L)	0.12	-	-	-	-	-	<0.005	<0.005	-
Ammonia-N (mg/L)	0.17	-	-	-	-	-	0.2	0.1	0.16
TKN (mg/L)	2.61	-	-	-	-	-	0.3	0.4	0.64
Metals and Pesticides									
Dissolved copper (µg/L)	6.4	-	-	-	-	-	4.6	3.6	-
Total copper (µg/L)	29.8	15	175	170	10	70	4.9	5.9	30
Dissolved lead (µg/L)	0.5	-	-	-	-	-	<0.07	<0.07	7.8
Total lead (µg/L)	8.6	6.1	53.5	95.2	7.6	36.8	1	0.8	-
Dissolved zinc (µg/L)	14	-	-	-	-	-	12	8.7	10
Total zinc (µg/L)	71	40	330	330	30	225	17.5	15	150

Constituent	LACDPW Mass Emission Station	NRSP Area Tributary Monitoring					Newhall WRP Startup Monitoring		USGS Wet Weather Monitoring
	S29	*Site A*	*Site B*	*Site C*	*Site D*	*Site E*	*NR1*	*NR3*	*USGS*
Dissolved aluminum (µg/L)	264	-	-	-	-	-	27	19	-
Total aluminum (µg/L)	5,770	-	-	-	-	-	740	770	-
Diazinon (µg/L)	0.05	-	-	-	-	-	<0.01	<0.01	0.02
Chlorpyrifos (µg/L)	<0.05	-	-	-	-	-	<0.6	<0.6	-
Cyanide (mg/L)	<0.01	-	-	-	-	-	-	-	-
Indicator Bacteria									
Fecal Coliform[3] MPN/100mL	101,000	3,300	590	4,200	>19,600	19,600	87	258	300
Total Coliform[3] MPN/100mL	7,000	38,700	>160,000	120,000	>89,400	>19,600	284	549	-

[1] Derived from Specific Conductance

[2] Nitrate + Nitrite-N

[3] Bacteria averages are represented as Geometric Means

ND = non detected, - = no or insufficient data

Table 2-23: Summary of Average Wet Weather Monitoring Data for 2-Day Precedent Rainfall of > 1 inch.

Constituent	USGS Wet Weather Monitoring	LACDPW SCR Mass Emission Station	Newhall WRP Startup Monitoring
	11108500	*S29*	*NR3*
General and Conventional Parameters			
TSS (mg/L)	10,711	1,482	43,360
TDS (mg/L)	838[1]	101	2,100
Hardness (mg/L)	546	197	832
Chloride (mg/L)	61	22	-
Nutrients			
Total P (mg/L)	1.02	0.54	13.4
Nitrate-N (mg/L)	1.7[2]	0.74	1.4
Nitrite-N (mg/L)		0.13	ND
Ammonia-N (mg/L)	-	0.23	0.5
TKN (mg/L)	0.69	4.32	46.0

Constituent	USGS Wet Weather Monitoring *11108500*	LACDPW SCR Mass Emission Station *S29*	Newhall WRP Startup Monitoring *NR3*
Metals			
Dissolved copper (µg/L)	-	8.4	-
Total copper (µg/L)	-	31.1	-
Dissolved lead (µg/L)	ND	2.4	-
Total lead (µg/L)	ND	29.9	-
Dissolved zinc (µg/L)	-	24	-
Total zinc (µg/L)	-	126	-
Dissolved aluminum (µg/L)	-	420	-
Total aluminum (µg/L)	-	5,161	-
Indicator Bacteria			
Fecal Coliform (MPN/100 mL)	2,700	36,000	≥1600
Total Coliform (MPN/100 mL)	-	198,000	≥1600

[1] Derived from Specific Conductance

[2] Nitrate + Nitrite-N

ND = Not Detected in Sample, - = no or insufficient data

2.6.2 Dry Weather Water Quality Monitoring

Dry season base flows in the SCR through the NRSP area are perennial. Dry season base flows may include contributions from natural groundwater flows; however, discharges from the upstream Saugus and Valencia WRPs contribute the majority of base flow. Discharges from the WRPs during dry weather conditions are a source of impairing pollutants in downstream reaches, including chloride, TDS, and nitrogen compounds.

Dry weather water quality monitoring data in the SCR are available from four sources (see Figure 2-1 for locations):

LACDPW sampling at the SCR mass emission station

USGS Water Quality Monitoring

Newhall Ranch WRP pre-startup monitoring (2004-2007)

Newhall Ranch NPDES monitoring (2008-2009)

These sites were described above under Wet Weather Monitoring (Section 2.3.1). The LACDPW station is on the SCR above Newhall Ranch, while the Newhall Ranch WRP pre-startup monitoring stations are at the western boundary and downstream of the NRSP area. The USGS station is also below the NRSP area, and provides a historical perspective from samples collected between 1951 and 1995.

General Constituents

Tables 2-24 through 2-26 report summary statistics for dry weather monitoring of selected nutrients in the three datasets.

Table 2-24: LACDPW Dry Weather Monitoring for Selected General Constituents at the SCR Mass Emission Station (S29) during 2002-2009

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
TSS (mg/L)	15	15	1	1320	135
Hardness (mg/L)	15	15	330	510	411
TDS (mg/L)	15	15	696	942	806

Table 2-25: Newhall WRP Pre-Startup Dry Weather Monitoring for Selected General Constituents in the SCR during 2004-2007

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
TSS (mg/L; DL = 1 mg/L)	NR1	98	97	<1	342	42
	NR3	98	97	<1	676	76
Hardness (mg/L)	NR1	98	98	150	568	323
	NR3	98	98	185	684	380
TDS (mg/L)	NR1	98	98	504	2,806	853
	NR3	98	98	576	1,396	930
Chloride (mg/L)	NR1	48	48	97	130	116
	NR3	48	48	102	140	122

Table 2-26: Newhall WRP NPDES Dry Weather Monitoring for Selected General Constituents in the SCR during 2008-2009

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
TSS (mg/L; DL = .5 mg/L)	RSW-001U	4	4	3.8[1]	36	18.1
	RSW-002D	4	4	2.7[1]	33	17.6
Hardness (mg/L)	RSW-001U	4	4	187	338	257
	RSW-002D	4	4	189	335	256
TDS (mg/L)	RSW-001U	4	4	694	1,028	873
	RSW-002D	4	4	870	950	904
Chloride (mg/L)	RSW-001U	4	4	111	120	115
	RSW-002D	4	4	114	135	122

[1] Analyte was detected at a concentration below the reporting limit and above the laboratory method detection limit. Reported value is estimated; detected, but not quantified (DNQ).

Table 2-27: USGS Dry Weather Water Quality Monitoring Data for Selected General Constituents in the SCR at the County Line during 1951-1995.

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
TSS (mg/L)	73	73	7	5,980	349
Hardness (mg/L)	220	220	42	2,400	881
Specific Conductance (uS/cm)	383	383	925	7,620	2408
Chloride (mg/L)	355	355	30	585	140

TSS. Average concentrations of TSS appeared highly variable between the monitoring stations. The USGS dataset showed relatively high average concentrations, which may have included samples taken during times of higher erosion or larger dry weather flows. Differences may also be due to physical factors such as substrate material, local flow regime, and tributary influences.

Hardness, TDS and Chloride. The average concentrations of dissolved constituents, hardness, TDS, and chloride were more similar between the monitoring locations and times. However, the USGS County Line station again consistently recorded higher averages (approximately double) than the other stations. The data suggests that the water flowing in the SCR in the NRSP area during dry weather is hard and turbid, with moderate levels of other dissolved salts, including chloride.

Nutrients

Tables 2-27 through 2-29 report summary statistics for dry weather monitoring of selected nutrients in the three datasets.

Table 2-28: LACDPW Dry Weather Monitoring of Nutrients at the SCR Mass Emission Station (S29) during 2002-2009

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved phosphorus (mg/L)	15	14	<0.05	0.30	0.18
Total phosphorus (mg/L)	15	14	<0.05	0.67	0.23
Nitrate-N (mg/L)	15	13	<0.17	1.78	1.16
Nitrite-N (mg/L)	15	2	<0.03	0.60	0.08
Ammonia-N (mg/L)	15	2	<0.03	0.81	0.08
TKN (mg/L)	15	15	0.23	1.31	0.60

- = no or insufficient data

Table 2-29: Newhall WRP Pre-Startup Dry Weather Monitoring for Selected Nutrients in the SCR during 2004-2007

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Total phosphorus (mg/L)	NR1	98	98	0.05	1.4	0.6
	NR3	98	97	<0.008	1.0	0.5
Nitrate-N (mg/L)	NR1	98	98	0.97	4.9	2.4
	NR3	98	97	<0.01	5.1	2.4
Nitrite-N (mg/L)	NR1	98	36	<0.005	0.2	<0.005
	NR3	98	31	<0.005	0.3	<0.005
Ammonia-N (mg/)	NR1	98	68	<0.005	0.4	0.1
	NR3	98	72	<0.005	0.4	0.1
TKN (mg/L)	NR1	95	89	<0.04	3.4	0.7
	NR3	95	91	<0.04	1.5	0.7

Table 2-30: Newhall WRP Pre-Startup Dry Weather Monitoring for Selected Nutrients in the SCR during 2008-2009

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Total phosphorus (mg/L)	RSW-001U	4	4	0.22	0.64	0.4
	RSW-002D	4	4	0.22	0.67	0.4
Nitrate-N (mg/L)	RSW-001U	4	4	1.41	2.33	1.8
	RSW-002D	4	4	1.29	2.45	1.9
Nitrite-N (mg/L)	RSW-001U	4	3	<0.01	0.21	0.07
	RSW-002D	4	3	<0.01	0.22	0.07
Ammonia-N (mg/)	RSW-001U	4	3	<0.03	0.06	0.04
	RSW-001D	4	2	<0.03	0.07	0.03
	RSW-002D	4	3	<0.03	0.1	0.05
TKN (mg/L)	RSW-001U	4	4	0.57[1]	0.98	0.77
	RSW-002D	4	3	<0.46	1.4	0.74

[1] Analyte was detected at a concentration below the reporting limit and above the laboratory method detection limit. Reported value is estimated; detected, but not quantified (DNQ).

Table 2-31: USGS Dry Weather Water Quality Monitoring Data for Selected Nutrients in the Santa Clara River at the County Line during 1951 - 1995

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved phosphorus (mg/L)	48	48	0.12	2.4	1
Total phosphorus (mg/L)	64	64	0.23	5.9	1.13
Ammonia as N (mg/L)	41	41	0.01	0.62	0.18
Nitrate + Nitrite as N (mg/L)	47	47	1.8	7.5	4
TKN as N (mg/L)	20	20	0.08	1.3	0.83
Total Nitrogen (mg/L)	33	33	0.5	15	3.7

Phosphorus and Nitrogen. The average concentrations for all nutrients showed a very similar and simple pattern. Concentrations generally increased downstream and were higher in the historical dataset. Lower average values at the mass emission station could reflect its location above the Valencia WRP, and/or the low number of dry weather samples at this station. Higher average concentrations at the USGS gauge (Table 2-29) compared with the Newhall WRP startup monitoring data (Table 2-28) could be due to greater nutrient loading over its period of record due to historically greater WRP discharge concentrations and/or less responsible use of fertilizers, as well as the higher TSS, and hence particulate nutrients, observed at this site.

Metals and Pesticides

Tables 2-30 through 2-32 report summary statistics for dry weather monitoring of selected metals and pesticides for the three datasets.

Table 2-32: LACDPW Dry Weather Monitoring for Metals and Pesticides at the SCR Mass Emission Station (S29) during 2002-2009

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved copper (µg/L)	15	15	0.99	3.8	2.4
Total copper (µg/L)	15	15	4.41	34	13
Dissolved lead (µg/L)	15	0	<0.17	<0.17	<0.17
Total lead (µg/L)	15	13	<0.17	8.2	1.3
Dissolved zinc (µg/L)	15	11	<1	26	8
Total zinc (µg/L)	15	13	<1	52	21
Dissolved cadmium (µg/L)	15	3	<0.05	41	3
Total cadmium (µg/L)	15	4	<0.05	72	5
Dissolved aluminum (µg/L)	15	0	<100	<100	<100
Total aluminum (µg/L)	15	3	<100	7500	566
Chlorpyrifos (µg/L)	15	0	<0.05	<0.05	<0.05
Diazinon (µg/L)	15	1	<0.01	0.23	<0.01

- = no or insufficient data

Table 2-33: Newhall WRP Pre-Startup Dry Weather Monitoring for Selected Metals, Pesticides in the SCR during 2004-2007.

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Dissolved copper (µg/L)	NR1	31	31	2.2	5.8	3.6
	NR3	31	31	2.3	5.5	3.6
Total copper (µg/L)	NR1	42	42	2.3	11	4.4
	NR3	42	42	2.1	15	5.2
Dissolved lead (µg/L)	NR1	31	8	<0.05	0.7	<0.05
	NR3	31	7	<0.05	0.6	<0.05
Total lead (µg/L)	NR1	42	38	<0.05	4.6	0.6
	NR3	42	39	<0.05	5.8	0.9
Dissolved zinc (µg/L)	NR1	31	31	7.8	22.2	14.1
	NR3	31	31	5.4	18.6	11.8

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Total zinc (µg/L)	NR1	42	42	8.5	30	16
	NR3	42	42	7.8	51	17
Dissolved aluminum (µg/L)	NR1	25	14	<5	290	36
	NR3	25	11	<5	750	54
Total aluminum (µg/L)	NR1	25	25	12	2100	325
	NR3	25	25	49	3300	530
Diazinon (µg/L)[1]	NR1	42	1	<0.01	2	<2
	NR3	42	2	<0.01	33.5	<2

[1] Detection limits changed over time.

Table 2-34: Newhall WRP Pre-Startup Dry Weather Monitoring for Selected Metals, Pesticides in the SCR during 2008-2009

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Total Copper (µg/L)	RSW-001U	4	4	1.5	3.7	2.9
	RSW-002D	4	4	1.4	3.9	2.9
Total Lead (µg/L)	RSW-001U	4	4	0.05[1]	0.54	0.29
	RSW-002D	4	4	0.06[1]	0.46	0.28
Total Zinc (µg/L)	RSW-001U	4	4	11.5	15.2	13.5
	RSW-002D	4	4	9.3	14.5	12.7
Total Aluminum (µg/L)	RSW-001U	4	4	21	427	207
	RSW-002D	4	4	21	386	170
Diazinon (µg/L)	RSW-001U	4	0	-	-	<0.002
	RSW-002D	4	0	-	-	<0.002

[1] Analyte was detected at a concentration below the reporting limit and above the laboratory method detection limit. Reported value is estimated; detected, but not quantified (DNQ).

Table 2-35: USGS Dry Weather Water Quality Monitoring Data for Selected Metals, Pesticides in the Santa Clara River at the County Line.

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Dissolved copper (ug/L)	40	13	1	5	1.8
Total copper (ug/L)	12	6	10	40	20
Dissolved lead (ug/L)	39	4	1	23	7.8
Total lead (ug/L)	30	0	ND	ND	ND
Dissolved Zinc (ug/L)	39	29	5	50	15.8
Total zinc (ug/L)	12	12	20	110	45

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Diazinon (ug/L)	6	4	0.01	0.05	0.03

ND = non detected

Metals. Concentrations of heavy metals in dry weather flows were generally low and, for the most part, reasonably similar. Total metal concentrations are generally controlled by TSS concentrations, and this is reflected in the difference between the historical data collected at the USGS site with high TSS and the more recent data with low TSS. Therefore, the dissolved concentrations are more interesting to compare. Average dissolved copper concentrations were fairly similar and ranged from 1.8 – 4.2 µg/L. Average dissolved zinc concentrations were also fairly similar and ranged from 11 – 24 µg/L. Higher copper and zinc concentrations were observed at the upper SCR site, which may reflect its proximity to urban land uses; however, the data are too few to confidently assert a reason for these differences. Dissolved lead showed some large differences between the historical and more recent datasets, and this is likely due to difficulties in analyzing trace metals in the earlier dataset, and widespread use of leaded gasoline prior to 1995.

Pesticides. Diazinon was detected at the upstream LACDPW site and historically at the USGS site. The more extensive data set collected at the Newhall WRP start-up sites did not detect diazinon and this may be due to its recent phase-out by USEPA for residential uses.

Fecal Indicator Bacteria

Tables 2-36 through 2-39 report summary statistics for dry weather monitoring of fecal indicator bacteria (FIB) for the three datasets. The concentrations of indicator bacteria indicated highly variable but generally elevated FIB concentrations in the SCR. Average concentrations of total coliform and fecal coliform were similar between the USGS station and the WRP startup monitoring stations. The mass emission station recorded much greater average concentrations, which is likely an artifact of the small dataset.

Table 2-36: LACDPW Dry Weather Monitoring at the SCR Mass Emission Station (S29) during 2002-2009

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Total coliform (MPN/100mL)	15	15	130	50000	3714
Fecal coliform (MPN/100mL)	15	15	20	5000	148
Enterococci (MPN/100mL)	15	14	<20	1300	140

Table 2-37: Newhall WRP Pre-Startup Dry Weather Monitoring for Indicator Bacteria in the SCR during 2004 – 2007

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Fecal coliform (CFU/100mL)	NR1	98	97	<2	2300	158
	NR3	98	97	<2	3000	187
Total coliform (MPN/100mL)	NR1	98	98	23	24,000	1227
	NR3	98	98	23	24,000	1452

Table 2-38: Newhall WRP Pre-Startup Dry Weather Monitoring for Indicator Bacteria in the SCR during 2008-2009

Constituent	Location	No. of Samples	No. of Detects	Min	Max	Average
Fecal coliform (MPN/100mL)	RSW-001U	4	3	<20	300	115
	RSW-002D	4	3	<20	170	92
Total coliform (MPN/100mL)	RSW-001U	4	3	<20	3000	1,827
	RSW-002D	4	3	<20	3000	982

Table 2-39: USGS Dry Weather Water Quality Monitoring Data for Indicator Bacteria in the Santa Clara River at the County Line during 1951-1995

Constituent	No. of Samples	No. of Detects	Minimum	Maximum	Average
Fecal coliform (CFU/100mL)	46	46	25	980	250

Summary

Table 2-40 summarizes of all dry weather monitoring data available for the Santa Clara River in the NRSP area.

Table 2-40: Summary of Average Dry Weather Monitoring Data in the Santa Clara River

Constituent	USGS Wet Weather Monitoring	SCR Mass Emission Station	Newhall WRP Startup Monitoring				
	11108500	*S29*	*NR1*	*NR3*	*RSW-001U*	*RSW-001D*	*RSW-002D*
General and Conventional Parameters							
TSS (mg/L)	349	135	42	76	18		18
Hardness (mg/L)	881	411	323	380	257		256
TDS (mg/L)	1541[1]	806	853	930	873		904
Chloride (mg/L)	140	114	116	122	115		122
Nutrients							
Total P (mg/L)	1.13	0.18	0.6	0.5	0.40		0.40
Nitrate-N (mg/L)	4[2]	0.23	2.4	2.4	1.84		1.86
Nitrite-N (mg/L)	-	1.16	<0.005	<0.005	0.07		0.07
Ammonia-N (mg/L)	0.18	0.08	0.1	0.1	0.04	0.03	0.05
TKN (mg/L)	0.83	0.08	0.7	0.7	0.77		0.74
Metals							
Dissolved copper (µg/L)	1.8	2.4	3.6	3.6			
Total copper (µg/L)	20	13	4.4	5.2	2.9		2.9
Dissolved lead (µg/L)	7.8	<0.17	<0.05	<0.05			
Total lead (µg/L)	ND	1.3	0.6	0.9	0.29		0.28
Dissolved zinc (µg/L)	15.8	7.9	14.1	11.8			
Total zinc (µg/L)	45	21	16	17	13.5		12.7
Dissolved aluminum (µg/L)	-	36	36	54			
Total aluminum (µg/L)	-	566	325	530	208		170

ND = non detected, - = no or insufficient data

2.7 Groundwater

2.7.1 Groundwater Beneficial Uses

The Project area is within the Basin Plan's Castaic Valley and Saugus Aquifer subbasin of the Santa Clarita Valley Groundwater Basin, East Subbasin. Beneficial uses for groundwaters for this subbasin are shown in Table 2-37.

Table 2-41: Beneficial Uses of Groundwaters

Groundwater Basin	MUN
DWR 4.07 - Eastern Santa Clara Sub-basin: Castaic Valley and Saugus Aquifer	E

E-Existing Beneficial Use
MUN: Community, military, or individual water supply systems including, but not limited to, drinking water supply
Source: Water Quality Control Plan for the Los Angeles Region (Basin Plan) (LARWQCB, 1994 as amended)

2.7.2 Existing Groundwater Quality

The Project area lies at the western end of the upper Santa Clara River hydrologic area, as defined by the California Department of Water Resources (DWR). The Santa Clara River Valley East Groundwater Subbasin lies within this hydrologic area and is the source of essentially all local groundwater used for water supply in the Santa Clarita Valley. The local groundwater supplies are obtained from relatively young surficial alluvial deposits and from an older geologic unit (the Saugus Formation) that underlies the alluvium and adjoining areas. The alluvium and the Saugus Formation are underlain by bedrock units consisting of the Pico Formation in the Project area and other geologic units in the eastern and northern portions of the Santa Clarita Valley. These deep bedrock units yield little water and are not considered viable for groundwater development.

The alluvial sediments lie within the portion of the Valley occupied by the Santa Clara River and also are present in side canyons that contain tributaries to the River. The alluvium consists of extensively interlayered and interfingered mixtures of gravel and sand, with variable amounts of cobbles and boulders and minor amounts of silt and clay. Due to the unconsolidated to poorly consolidated condition of the alluvium, and its lack of cementation, the alluvium has relatively high permeability and porosity. The groundwater flow direction in the Alluvial aquifer follows the topography of the Valley and its tributaries. Groundwater recharge occurs in the eastern, northern, and southern portions of the Valley. Natural mechanisms for groundwater discharge occur at the west end of the Valley and consist of discharge to the Santa Clara River, subsurface outflow beneath the River, and evapotranspiration by deep-rooted vegetation.

The Saugus Formation is present beneath the Project and most of the Santa Clarita Valley area east of the NRSP area. The upper subunits of the Saugus Formation consist of terrestrial sediments deposited in stream channels, floodplains, and alluvial fans by ancestral drainage systems. The upper subunits are a source of groundwater supply in the Santa Clarita Valley because of their productive nature and their good water quality. Deeper subunits of the Saugus

Formation were deposited in a marine environment and are subsequently not used for water supplies because of their brackish water quality and fine-grained, low-permeability nature.

Faulting and folding of the Saugus Formation and the underlying bedrock units have created a bowl-shaped structure beneath the Santa Clarita Valley. The Saugus Formation and underlying bedrock generally dip downwards from the periphery of the Valley towards the deepest portion of the "bowl" beneath the central portion of the Valley. The thickness of the Saugus Formation also is controlled by the San Gabriel fault, which is present in the eastern and northern portions of the Valley. Because of its structure and its connection with the overlying Alluvial aquifer, groundwater flow in the Saugus Formation is generally towards the center of the bowl and also towards the western portion of the Santa Clara River. Like the Alluvial aquifer, the Saugus Formation is recharged in the eastern and other peripheral portions of the Santa Clarita Valley. Groundwater discharge from the Saugus Formation occurs at the west end of the Valley in the form of groundwater discharge into the overlying Alluvial aquifer, which in turn discharges to the River in the western end of the Valley.

Alluvium. In terms of the aquifer system, there is no convenient long-term record of water quality (*i.e.*, water quality data in one or more single wells that spans several decades and continues to the present). Thus, in order to examine a long-term record of water quality in the alluvium, individual records have been integrated from several wells completed in the same aquifer materials and in close proximity to each other to examine historical trends in general mineral groundwater quality throughout the basin (Luhdorff & Scalmanini, 2005). Based on these records of groundwater quality, wells within the alluvium have experienced historical fluctuations in general mineral content, as indicated by electrical conductivity (EC), which correlates with fluctuations of individual constituents that contribute to EC. However, the historic water quality data indicates that, on a long-term basis, there has not been a notable trend and, specifically, there has not been a decline in water quality within the alluvium.

Specific conductance within the alluvium exhibits a westward gradient, corresponding with the direction of groundwater flow in the alluvium. EC is lowest in the easternmost portion of the basin, and highest in the west, and generally exhibits an inverse correlation with precipitation and streamflow, with a stronger correlation in the easternmost portion of the basin where groundwater levels fluctuate the most. Wet periods have produced substantial recharge of higher quality (low EC) water, and dry periods have resulted in declines in groundwater levels, with a corresponding increase in EC (and individual contributing constituents) in the deeper parts of the alluvium.

The most notable groundwater quality issue in the alluvium is perchlorate contamination in a localized area situated about three miles east of the Project area. In 2002, one well (the Santa Clarita Water Division's Stadium Well), located near the former Whittaker-Bermite facility, was inactivated for municipal water supply due to detection of perchlorate slightly below the Notification Level. Since that time, a replacement well (Valley Center Well) has been drilled in

the non-impacted portion of the groundwater basin. In early 2005, perchlorate was detected in a second well, the Valencia Water Company's Well Q2. In October 2005, Well Q2 was returned to service with wellhead perchlorate treatment under a permit from the California DHS. On-going monitoring in the alluvium north of the Whittaker-Bermite site (an ammunition manufacturing site) has shown no detections of perchlorate in any other Alluvial municipal water supply wells in this area.

Table 2-42 summarizes average metals, general chemistry, and organic compounds data for three Alluvial aquifer wells located in and near the Project area (see Figure 2-1). One well is a municipal water supply well that belongs to the Valencia Water Company (E-15) and is located in the Valencia Commerce Center area, northeast of the Project boundary. Two Newhall Ranch agricultural Alluvial aquifer wells (C and B6) were monitored twice (once each in 2000 and 2001).

Laboratory testing indicates that all constituents tested were at acceptable levels for drinking water, for all tested wells, with the exception of sulfate and iron in the agricultural supply well B6. Specifically, the average sulfate concentration (360 mg/L) exceeded the Basin Plan objective of 350 mg/L and the average iron concentration (0.4 mg/L) exceeded the secondary drinking water standard of 0.3 mg/L in Alluvial Well B6.

Tests conducted for perchlorate at the Alluvial aquifer wells listed in Table 2-42 indicated "non-detect," meaning no perchlorate was detected. Furthermore, no organic contaminants have been detected in any Alluvial aquifer wells.

Saugus Formation. Similar to the Alluvial aquifer, groundwater quality in the Saugus Formation is a key factor in assessing that aquifer as a municipal and agricultural water supply. As with the Alluvial aquifer, long-term Saugus groundwater quality data is not sufficiently extensive (few wells) to permit any basin-wide analysis or assessment of pumping-related impacts on quality. Accordingly, EC has been chosen as an indicator of overall water quality, and records have been combined to produce a long-term depiction of water quality. Water quality in the Saugus Formation historically has not exhibited the precipitation-related fluctuations seen in the Alluvial aquifer, and based on the historical record over the last 50 years, groundwater quality in the Saugus Formation has exhibited a slight overall increase in EC.

Table 2-42 summarizes average metals, general chemistry, and organic compounds data for one Saugus aquifer well located near the Project location (see Figure 2-1). Saugus Well 206 is a municipal water supply well that belongs to the Valencia Water Company and is located in the RMDP project area. Laboratory testing indicates that all constituents tested were at acceptable levels for drinking water in Saugus Well 206.

As with the Alluvial aquifer, the most notable groundwater quality issue in the Saugus Formation is perchlorate contamination. Since 1997, five Saugus wells located near the former Whittaker-

Bermite facility (about two miles east of the Project location), including the recently closed Valencia Water Company (VWC) Well 201, have been inactivated for water supply service for varying periods of time due to the presence of perchlorate. Since that time, the Department of Public Health approved a Final Interim Remedial Action Plan for the containment and extraction of perchlorate from the groundwater, and the Castaic Lake Water Agency (CLWA), in partnership with other local retail purveyors and the City of Santa Clarita, completed construction of CLWA's Saugus Perchlorate Treatment Facility, a $13-million facility to treat perchlorate in groundwater emanating from the Whittaker-Bermite site. As a result, two of the previously inactivated Saugus wells have been placed back into service, and a replacement well for a third well has been drilled in the non-impacted portion of the groundwater basin. The Saugus wells, in combination with the reinstated Alluvial wells, collectively restore much of the temporarily lost well capacity, and an additional two wells will be drilled to restore the operational flexibility that existed prior to the detection of perchlorate. Specific to Well 201, VWC plans to actively seek remediation and restore the impacted well capacity in the near term.

Table 2-42: Groundwater Monitoring Data

Parameter	Units	Basin Plan Objective / Maximum Contaminant Level	Average Concentration			
			Alluvial Well E-15	Alluvial Well C	Alluvial Well B6	Saugus Well 206
Aluminum	µg/L	1,000[2]	ND	ND	ND	ND
Arsenic	µg/L	50[2]	n/a	ND	ND	n/a
Barium	mg/L	1[2]	ND	0.02	0.03	ND
Beryllium	µg/L	4[2]	ND	n/a	n/a	ND
Cadmium	µg/L	5[2]	ND	ND	ND	ND
Chromium	µg/L	50[2]	ND	ND	ND	ND
Copper	µg/L	1,000[3]	ND	ND	ND	ND
Iron	mg/L	0.3[3]	ND	0.1	**0.4**	ND
Manganese	µg/L	50[3]	ND	ND	ND	ND
Mercury, Total	µg/L	2[2]	n/a	ND	ND	n/a
Nickel	µg/L	100[2]	ND	ND	ND	ND
Selenium	µg/L	50[2]	n/a	ND	ND	n/a
Silver	µg/L	100[3]	NA	ND	ND	n/a
Thallium	µg/L	2[2]	NA	ND	ND	n/a
Zinc	µg/L	5,000[3]	ND	ND	ND	ND
Alkalinity as CaCO3	mg/L	--	226	255	295	221
Boron	mg/L	1.0[1]	0.48	0.39	0.48	n/a
Chloride	mg/L	150[1]	90	57	82	45
Color	Color unit	15[3]	ND	ND	5	ND
Cyanide, total	mg/L	0.15[2]	n/a	ND	ND	n/a
Fluoride	mg/L	2.0[2]	0.8	0.7	0.8	0.2
Hardness as CaCO3	mg/L	--	499	410	510	464
MBAS	mg/L	0.5[3]	n/a	ND	ND	n/a
Nitrate as NO3	mg/L	45[1]	18.5	9.5	10.6	20.9
Nitrite as N	mg/L	1[1]	ND	ND	ND	ND
Nitrate+Nitrite as N	mg/L	10[1]	3.6	2.1	2.4	4.7
Odor	TON	3[3]	1.1	ND	ND	1

Parameter	Units	Basin Plan Objective / Maximum Contaminant Level	Average Concentration			
			Alluvial Well E-15	Alluvial Well C	Alluvial Well B6	Saugus Well 206
Specific Conductance	umhos/cm	900-1600[3]	1317	1150	1400	1158
Sulfate	mg/L	350[1]	314	285	**360**	293
TDS	mg/L	1,000[1]	969	760	950	861
Turbidity	NTU	5[3]	0.4	0.35	1.4	0.2
Volatile Organic Chemicals (VOCs)	µg/L	variable	ND	ND	ND	ND
Synthetic Organic Chemicals (SVOCs)	µg/L	variable	ND	ND	ND	ND
Key: **Bold** **Exceeds Standard**						

-- = no applicable basin plan objective or MCL
n/a = not analyzed
ND = none detected
[1] Los Angeles Basin Plan Regional Objectives for Groundwater (Table 3-10).
[2] California Department of Public Health Primary Drinking Water MCL (Title 22 CCR Table 64431-A and Table 64444-A).
[3] California Department of Public Health Secondary Drinking Water MCL (Title 22 CCR Table 64449-A and Table 64449-B).

3 REGULATORY SETTING

3.1 Clean Water Act

In 1972, the Federal Water Pollution Control Act [later referred to as the Clean Water Act (CWA)] was amended to require National Pollutant Discharge Elimination System (NPDES) permits for the discharge of pollutants to waters of the United States from any point source. In 1987, the CWA was amended to require that the United States Environmental Protection Agency (USEPA) establish regulations for permitting of municipal and industrial stormwater discharges under the NPDES permit program. The USEPA published final regulations regarding stormwater discharges on November 16, 1990. The regulations require that municipal separate storm sewer system (MS4) discharges to surface waters be regulated by a NPDES permit.

In addition, the CWA requires the States to adopt water quality standards for receiving water bodies and to have those standards approved by the USEPA. Water quality standards consist of designated beneficial uses for a particular receiving water body (e.g. wildlife habitat, agricultural supply, fishing etc.), along with water quality criteria necessary to support those uses. Water quality criteria are prescribed concentrations or levels of constituents – such as lead, suspended sediment, and fecal coliform bacteria – or narrative statements which represent the quality of water that support a particular use. Because California had not established a complete list of acceptable water quality criteria, USEPA established numeric water quality criteria for certain toxic constituents in receiving waters with human health or aquatic life designated uses in the form of the California Toxics Rule ("CTR") (40 CFR 131.38).

3.2 CWA Section 303(d) - TMDLs

When designated beneficial uses of a particular receiving water body are being compromised by water quality, Section 303(d) of the CWA requires identifying and listing that water body as "impaired". Once a water body has been deemed impaired, a Total Maximum Daily Load (TMDL) must be developed for the impairing pollutant(s). A TMDL is an estimate of the total load of pollutants from point, non-point, and natural sources that a water body may receive without exceeding applicable water quality standards (with a "factor of safety" included). Once established, the TMDL allocates the loads among current and future pollutant sources to the water body.

The Project would discharge runoff to Santa Clara River Reach 5. Table 3-1**,** 2010 CWA Section 303(d) Listings for the Santa Clara River Mainstem, lists the water quality impairments for the Santa Clara River mainstem, as reported in the 2010 CWA Section 303(d) list of water quality limited segments, including reaches upstream of the Project location. Reach 7 of the Santa Clara River (Bouquet Canyon Road to above Lang Gaging Station) is listed for coliform bacteria. Reach 6 (West Pier Highway 99 to Bouquet Canyon Road) is listed for coliform bacteria, chlorpyrifos, diazinon, toxicity, iron, and copper. Reach 5 is listed for coliform bacteria and iron.

Santa Clara River Reach 3, approximately 25 miles downstream of the Project location and below the Dry Gap in Reach 4, is listed for total dissolved solids (TDS) and toxicity. Santa Clara River Reach 1, approximately 30 miles downstream of the Project location, is listed for toxicity. The Santa Clara River estuary, located approximately 40 miles downstream of the Project location, is listed for coliform, chlorinated legacy pesticides, Toxaphene, toxicity, and nitrate-nitrogen.

The RWQCB has adopted nitrogen compounds (nitrate-nitrogen plus nitrite-nitrogen and ammonia), chloride, and indicator bacteria TMDLs in the Basin Plan. Table 3-2, 2010 CWA Section 303(d) List of Water Quality Limited Segments Being Addressed By EPA Approved TMDLs, lists the 2010 Section 303(d) List of Water Quality Limited Segments Being Addressed by EPA Approved TMDLs. The Indicator Bacteria TMDL, adopted by the Regional Board on July 8, 2010, must be submitted for review and approval to the SWRCB, the State Office of Administrative Law, and the USEPA. The wasteload allocations for stormwater discharges into Reach 5 of the Santa Clara River are summarized in Table 3-3, TMDL Wasteload Allocations for MS4 and Stormwater Sources to Santa Clara River Reach 5. Pollutant reductions are regulated through effluent limits prescribed in Publicly Owned Treatment Works (POTW) and minor point source NPDES Permits, BMPs required in NPDES MS4 Permits, and SWRCB Management Measures for non-point source discharges.

3.3 California Toxics Rule

The California Toxics Rule (40 C.F.R. §131.38) is a federal regulation issued by the USEPA that provides water quality criteria for toxic pollutants in waters with human health or aquatic life designated uses in California. Not all waters receiving flows from the NRSP area, such as the tributaries to the Santa Clara River, are specifically designated with human health or aquatic life uses. However, the Santa Clara River does have such designated uses, and the impact analysis in Section 7 of this report assumes that the Santa Clara River Reach 5 beneficial uses apply to all of the Project's receiving waters pursuant to the Basin Plan. Although CTR criteria do not apply directly to discharges of stormwater runoff, they can provide a useful benchmark to assess the potential impacts to the water quality of receiving waters from Project stormwater runoff discharges. Here, the freshwater aquatic life criteria are used as benchmarks to evaluate the potential impacts of stormwater runoff to the project's receiving waters. The CTR also contains human health criteria which are derived for drinking water sources and for fish consumption only. Since the human health criteria are less stringent than the aquatic life criteria for the pollutants of concern for the Project, the aquatic life criteria are used.

Table 3-1: 2010 CWA Section 303(d) Listings for the Santa Clara River Mainstem

River Reach or Tributary	Geographic Description and Distance from Project to Upstream End of Reach	Pollutants	303(d) List Proposed TMDL Completion	Potential Sources
7	Bouquet Canyon Rd to above Lang Gaging Station (5 miles upstream)	Coliform Bacteria	2019[1]	Nonpoint and Point Sources
6	West Pier Hwy 99 to Bouquet Cyn Rd (Directly upstream of Project site)	Coliform Bacteria Chlorpyrifos Diazinon Toxicity Iron Copper	2019[2] 2019 2019 2019 2021 2021	Source Unknown Nonpoint and Point Sources Source Unknown] Source Unknown Source Unknown Nonpoint and Point Sources
5	Blue Cut Gaging Station to West Pier Hwy 99 (Project location)	High Coliform Count Iron	2019[1] 2021	Nonpoint and Point Sources Source Unknown
3	Freeman diversion dam to "A" street (25 miles)	Total Dissolved Solids Toxicity	2015 2021	Nonpoint and Point Sources Source Unknown
1	Estuary to Highway 101 Bridge (30 miles)	Toxicity	2019	Source Unknown
--	Estuary (40 miles)	ChemA[3] Coliform Toxaphene Toxicity Nitrate	2019 2019 2019 2019 2021	Source Unknown Nonpoint Source Nonpoint Source Source Unknown Source Unknown

[1] Reach 3 is downstream of the Dry Gap in Reach 4.
[2] Indicator Bacteria TMDL adopted by LARWQCB in July 2010; not yet approved by SWRCB and USEPA.
[3] ChemA suite of chlorinated legacy pesticides include: Aldrin, chlordane, Dieldrin, Endosulfan I/II, Endrin, gamma-BHC, heptachlor, heptachlor epoxide, and Toxaphene.

Table 3-2: 2010 CWA Section 303(d) List of Water Quality Limited Segments Being Addressed By EPA Approved TMDLs

Water Body Name	Pollutants	Potential Sources	US EPA Approved TMDL
Santa Clara River Reach 6	Ammonia Chloride	Nonpoint/Point Source Nonpoint/Point Source	2004 2005
Santa Clara River Reach 5	Chloride	Nonpoint/Point Source	2005
Santa Clara River Reach 3	Ammonia Chloride	Nonpoint/Point Source Nonpoint/Point Source	2002 2002

Table 3-3: TMDL Wasteload Allocations for MS4 and Stormwater Sources to Santa Clara River Reach 5

<table>
<tr><th>Impairing Pollutant</th><th colspan="3">Numeric Water Quality Objective</th><th>Wasteload Allocation</th></tr>
<tr><td>Chloride</td><td colspan="3">100 mg/L.</td><td>Wasteload allocations have been adopted for the Saugus WRP and the Valencia WRP. Other NPDES discharges contribute a minor chloride load. The wasteload allocation for these point sources is 100 mg/L.
The source analysis indicates that non-point sources are not a major source of chloride. The load allocations for non-point sources is 100 mg/L.</td></tr>
<tr><td rowspan="5">Nitrogen Compounds</td><td colspan="3">The numeric target for nitrogen in this TMDL is based on achieving the existing nitrogen water quality objective of 5 mg/L NO_3-N + NO_2-N. (Note: the numeric target that is used to calculate the wasteload allocations includes a 10% margin of safety; thus the numeric target is 4.5 mg/L NO_3-N + NO_2-N.)
The water quality objective for ammonia in Reach 5 used in the nitrogen compounds TMDL was based on temperature and pH for different River segments within the reach:</td><td rowspan="5">Concentration-based wasteloads are allocated to municipal, industrial and construction stormwater sources regulated under NPDES permits. For stormwater permittees discharging into Reach 5, the following wasteload allocations apply:
30-day average nitrate plus nitrite = 6.8 mg/L (NO_3-N+NO_2-N)
1-hour average ammonia = 5.2 mg/L (NH_3 as N)
30-day average ammonia = 1.75 mg/l (NH_3 as N)</td></tr>
<tr><td colspan="3">Ammonia Water Quality Objective (mg/L as N)[1]</td></tr>
<tr><td></td><td>1-hour average</td><td>30-day average</td></tr>
<tr><td>Reach 5 at County Line</td><td>3.4</td><td>1.2</td></tr>
<tr><td>Reach 5 below Valencia</td><td>5.5</td><td>2.0</td></tr>
<tr><td>Reach 5 above Valencia</td><td>4.8</td><td>2.0</td></tr>
</table>

<table>
<tr><th>Impairing Pollutant</th><th>Numeric Water Quality Objective</th><th>Wasteload Allocation</th></tr>
<tr>
<td>Indicator Bacteria (Resolution R10-006)</td>
<td>
Numeric Targets:
<table>
<tr><th>Constituent</th><th>SCR Reach 5 Requirement</th></tr>
<tr><td>E. Coli
(Single Sample)</td><td>235/100 mL</td></tr>
<tr><td>E. Coli
(Geometric Mean)</td><td>126/100 mL</td></tr>
</table>
</td>
<td>
Wasteload allocations are given in terms of allowable exceedance days. The numeric targets may not be exceeded more than the number of allowable exceedance days allotted in the tables below.
Interim Allowable Exceedance Days
(Enforceable 4 years after effective date of TMDL):
<table>
<tr><th>Time Period</th><th>Santa Clara River Reach 5</th></tr>
<tr><td>Dry Weather</td><td>17 allowable exceedance days of singe sample objectives.</td></tr>
<tr><td>Wet Weather</td><td>61 allowable exceedance days of singe sample objectives;</td></tr>
</table>
Allowable Exceedance Days
(Dry Weather enforceable 11 years after effective date of TMDL; Wet Weather enforceable 17 years after effective date of TMDL):
<table>
<tr><th>Time Period</th><th>Santa Clara River Reach 5</th></tr>
<tr><td>Dry Weather</td><td>5 allowable exceedance days of singe sample objectives:
0 allowable exceedances of geometric mean objectives</td></tr>
<tr><td>Wet Weather</td><td>16 allowable exceedance days of singe sample objectives;
0 allowable exceedances of geometric mean objectives</td></tr>
</table>
</td>
</tr>
</table>

[1] The numeric targets are 10 percent smaller to incorporate a margin of safety.

Freshwater aquatic life criteria for certain metals in the CTR are expressed as a function of hardness because hardness, and/or water quality characteristics that are usually correlated with hardness, can reduce the toxicities of some metals [5]. The minimum wet weather hardness value of 250 mg/L as $CaCO_3$ from USGS station 11108500 was used to approximate CTR criteria for metals. This value is likely to be more representative of conditions in the Santa Clara River within the NRSP subregion than Los Angeles County's Station 29 based on the water quality data summarized in Section 2.7 above. As per requirements of their discharge permit, the Valencia WRP has a monitoring station just upstream of the NRSP subregion area. Monthly hardness values for the Santa Clara River at this station ranged from 326 to 360 mg/L as $CaCO_3$ in 2004. Other water quality comparisons to this station were not made due to lack of wet weather monitoring. The hardness value of 250 mg/L is a conservative estimate of wet weather hardness values that should occur in the NRSP subregion area, although higher values are likely to occur.

The CTR also establishes two types of aquatic life criteria: acute and chronic. Acute criteria represent the highest concentration of a pollutant to which aquatic life can be exposed for a short period of time without deleterious effects; chronic criteria equal the highest concentration to which aquatic life can be exposed for an extended period of time (four days) without deleterious effects. Due to the intermittent nature of stormwater runoff (especially in southern California), the acute criteria are considered to be more applicable to stormwater conditions than chronic criteria. For example, the average storm duration in the 38-year Newhall gage rainfall record is 11.3 hours. In this document, the acute CTR criteria are used as one type of benchmark to evaluate the potential ecological impacts of Project runoff on the receiving waters.

3.4 California Porter-Cologne Act

The federal CWA places the primary responsibility for the control of surface water pollution and for planning the development and use of water resources with the states, although it does establish certain guidelines for the states to follow in developing their programs and allows USEPA to withdraw control from states with inadequate implementation mechanisms.

[5] The toxicity of a chemical to an aquatic organism may vary according to attributes of the organism, chemical composition, and exposure environment, so that the chemical is more or less "bioavailable." Many chemicals exist in a variety of forms (chemical species), and such chemical speciation affects bioavailability because relative uptake rates can differ among chemical species and the relative concentrations of chemical species can differ among exposure conditions. Usually, metal toxicity is reduced by increased water hardness, which is composed of cations (primarily calcium and magnesium). In some cases, the apparent effect of hardness on toxicity might be partly due to complexation of the metal by higher concentrations of hydroxide and/or carbonate (increased pH and alkalinity) commonly associated with higher hardness. (USEPA, 2007a)

California‘s primary statute governing water quality and water pollution issues with respect to both surface waters and groundwater is the Porter-Cologne Water Quality Control Act of 1970 (Porter-Cologne Act). The Porter-Cologne Act grants the State Water Resource Control Board (SWRCB) and the Regional Water Quality Control Boards (RWQCBs) power to protect water quality and is the primary vehicle for implementation of California’s responsibilities under the federal Clean Water Act. The Porter-Cologne Act grants the SWRCB and the RWQCBs authority and responsibility to adopt plans and policies to regulate discharges of waste to surface and groundwater, to regulate waste disposal sites, and to require cleanup of discharges of hazardous materials and other pollutants. The Porter-Cologne Act also establishes reporting requirements for unintended discharges of any hazardous substance, sewage, or oil or petroleum product.

Each RWQCB must formulate and adopt a water quality control plan (Basin Plan) for its region. The Basin Plan must conform to the policies set forth in the Porter-Cologne Act and established by the SWRCB in its state water policy. To implement state and federal law, the Basin Plan establishes beneficial uses for surface and groundwaters in the region, and sets forth narrative and numeric water quality standards to protect those beneficial uses. The Porter-Cologne Act also provides that a RWQCB may include within its regional plan water discharge prohibitions applicable to particular conditions, areas, or types of waste.

3.5 Basin Plan

The applicable Basin Plan (LARWQCB, 1994, as amended) provides numeric and narrative criteria for a range of water quality constituents applicable to certain receiving water bodies and groundwater basins within the Los Angeles region. Specific criteria are provided for the larger, designated water bodies within the region, as well as general criteria or guidelines for ocean waters, bays and estuaries, inland surface waters, and groundwaters. Those waters not specifically listed (generally smaller tributaries) are assumed to have the same beneficial uses as the streams, lakes, or reservoirs to which they are tributary. In general, the narrative criteria require that degradation of water quality does not occur due to increases in pollutant loads that will adversely impact the designated beneficial uses of a water body. For example, the Los Angeles Basin Plan requires that “Inland surface waters shall not contain suspended or settleable solids in amounts which cause a nuisance or adversely affect beneficial uses as a result of controllable water quality factors.” Water quality criteria apply within receiving waters as opposed to applying directly to runoff; therefore, water quality criteria from the Basin Plan are utilized as benchmarks as one method to evaluate the potential ecological impacts of Project runoff on the receiving waters of the proposed Project. Table 2-2 above lists the beneficial uses of applicable receiving surface waters.

The Basin Plan also contains water quality criteria for groundwater basins. For example, the Basin Plan requires that “Groundwaters shall not contain taste or odor producing

substances in concentrations that cause nuisance or adversely affect beneficial uses." Table 2-37 above lists the beneficial uses of the applicable groundwater basin.

3.6 MS4 Permit

In 2001, the Los Angeles Regional Water Quality Control Board (LARWQCB, 2001) issued an NPDES Permit and Waste Discharge Requirements (Order No. 01-182) under the CWA and the Porter-Cologne Act for discharges of urban runoff in public storm drains in Los Angeles County. The Permittees are the Los Angeles County cities and the County (collectively "the Co-Permittees"). This permit regulates stormwater discharges from MS4s in the Project area. The NPDES permit details requirements for new development and significant redevelopment, including specific sizing criteria for treatment BMPs and flow control requirements.

To implement the requirements of the NPDES permit, the Co-permittees have established development planning guidance and control measures that control and mitigate stormwater quality and quantity impacts to receiving waters as a result of new development and redevelopment. They are also required to implement other municipal source detection and elimination programs, as well as maintenance measures.

3.6.1 Stormwater Quality Management Program

The MS4 Permit contains the following provisions for implementation of the Stormwater Quality Management Program (SQMP) by the Co-permittees:

- General Requirements – Each Permittee is required to implement the SQMP to comply with applicable storm water program requirements and implement additional controls where necessary to reduce the discharge of pollutants in stormwater to the "maximum extent practicable" (MEP).
- BMP Implementation – Permittees are required to implement the most effective combination of BMPs for stormwater/urban runoff pollution control.
- SQMP Revision – Permittees are required to revise the SQMP to comply with regional, watershed specific requirements, and/or wasteload allocations for implementation of TMDLs for impaired waterbodies.
- Responsibilities of the Principal Permittee – The responsibilities of the Los Angeles County Department of Public Works (as the Principal Permittee) include, but are not limited to, coordinating activities necessary to comply with the NPDES permit, providing personnel and fiscal resources for SQMP updates and annual reports and summaries of reports required under the SQMP, and implementing and evaluating results of a County-wide Monitoring Program.

- Responsibilities of Permittees – Each Permittee is required to comply with the requirements of the SQMP applicable to the discharges within its boundaries.
- Watershed Management Committees (WMCs) – WMCs are comprised of a voting representative from each Permittee within the Watershed Management Areas (WMAs). WMCs are required to facilitate efforts and exchange of information between Permittees, establish additional goals for WMAs, prioritize pollution control efforts, monitor implementation of tasks designated for the WMA, and assess the effectiveness of and recommend revisions to the SQMP.
- Legal Authority – Permittees are granted the necessary legal authority to prohibit non-storm water discharges to the storm drain system.
- The objective of the SQMP is to reduce pollutants in urban stormwater discharges to the "maximum extent practicable" in order to attain water quality objectives and to protect the beneficial uses of receiving waters in Los Angeles County. Special provisions are provided in the MS4 permit to facilitate implementation of the SQMP. These provisions include:
- BMP substitution – Substitution of site-specific BMPs is allowed provided the alternative BMP will meet or exceed pollutant reduction of the original BMP, the fiscal burden of the original BMP is substantially greater than the proposed alternative, and the alternative BMP will be implemented within a similar time period.
- Public Information and Participation Program (PIPP) – This requires the Permittee to identify how public education needs were determined, who is responsible for developing and implementing the program, and the method used to determine its effectiveness.
- Industrial/Commercial Facilities Control Program – This requires the Permittee to develop a plan for managing stormwater runoff from industrial and commercial facilities. This program will track, inspect, and ensure compliance at industrial and commercial facilities that are sources of pollutants in storm water.
- Development Planning Program – This requires the Permittee to implement a development-planning program that requires new development and redevelopment projects to minimize impacts from stormwater and urban runoff.
- Development Construction Program – This requires the Permittee to implement a program to control runoff from construction activity to minimize erosion and transportation of sediment and prevent non-stormwater discharges from equipment and vehicle washing.
- Public Agency Activities Program – This requires municipalities to evaluate existing public agency activities that have an impact on stormwater quality (such as vehicle maintenance, landscape maintenance and weed control, and

construction and maintenance of streets, roads, and flood control systems) and to develop a program to reduce stormwater impacts with a schedule for implementation.

- Illicit Connections and Illicit Discharges Elimination Program – This requires each Permittee to have a plan for finding and preventing illegal connections and discharges and a mechanism for enforcing against illegal connections and discharges.

3.6.2 Standard Urban Stormwater Mitigation Plan

On March 8, 2000, the development planning program requirements, including the Standard Urban Stormwater Mitigation Plan requirements (collectively, development planning program requirements, including Standard Urban Stormwater Mitigation Pan requirements, are referred to in this report as SUSMP requirements) were approved by the RWQCB as part of the MS4 program to address stormwater pollution from new construction and redevelopment. The SUSMP contains a list of minimum BMPs that must be employed to infiltrate or treat stormwater runoff, control peak flow discharge, and reduce the post-project discharge of pollutants from stormwater conveyance systems. The SUSMP defines, based upon land use type, the types of practices that must be included and issues that must be addressed as appropriate to the development type and size. Compliance with SUSMP requirements is used as one method to evaluate significance of project development impacts on surface water runoff.

Finalized in May 2000, the County of Los Angeles' "Manual for the Standard Urban Stormwater Mitigation Plan" details the requirements for new development and significant redevelopment BMPs (Los Angeles County, 2000) (the "SUSMP Manual"). The SUSMP Manual is a model guidance document for use by Permittees and individual project owners to select post-construction BMPs and otherwise comply with the SUSMP requirements. It addresses water quality and drainage issues by specifying design standards for structural or treatment control BMPs that infiltrate or treat stormwater runoff and control peak flow discharge. BMPs are defined in the SUSMP Manual and SUSMP requirements as any program, technology, process, sizing criteria, operational methods or measures, or engineered systems, which, when implemented, prevent, control, remove, or reduce pollution. Treatment BMP sizing criteria and design guidance are also contained in the MS4 Permit and in the SUSMP Manual.

One of the most important requirements within the SUSMP is the specific sizing criteria for stormwater treatment BMPs for new development and significant redevelopment projects. The SUSMP includes sizing criteria for both volume-based and flow-based BMPs. The sizing criteria options for volume-based BMPs, such as extended detention basins, are as follows:

- The 85th percentile 24-hour runoff event storm event determined as the maximized capture stormwater volume for the area, from the formula recommended in Urban Runoff Quality Management, Water Environment Federation (WEF) Manual of Practice No. 23/ASCE Manual of Practice No. 87 (WEF, 1998);

- The volume of annual runoff based on unit basin storage volume, to achieve 80% or more volume treatment by the method recommended in California Stormwater Best Management Practices Handbook – Industrial/Commercial (SWQTF, 1993);

- The volume of runoff produced from a 0.75 inch storm event, prior to its discharge to a stormwater conveyance system; or

- The volume of runoff produced from a historical-record based reference 24-hour rainfall criterion for "treatment" (0.75 inch average for the Los Angeles County Area) that achieves approximately the same reduction in pollutant loads and flows as achieved by mitigation of the 85th percentile, 24-hour runoff event.

Stormwater treatment facilities will be designed to meet or exceed the sizing standards contained in the SUSMP Manual. Volume-based treatment control BMPs for the Project will be sized to capture and treat 80 percent of the annual runoff volume, with a drawdown time of 48 hours. This methodology utilizes historical rainfall data with continuous simulation modeling to calculate the treatment volume for each treatment control BMP and is consistent with criteria 2 above.

Flow-based BMPs, such as vegetated swales, must be designed to infiltrate or treat the maximum flow rate generated from one of the following scenarios:

- The flow of runoff produced from a rain event equal to at least 0.2 inches per hour intensity,

- The flow of runoff produced from a rain event equal to at least two times the 85th percentile hourly rainfall intensity for Los Angeles County, or

- The flow of runoff produced from a rain event that will result in treatment of the same portion of runoff as treated using volumetric standards above.

Flow-based BMPs for the Project will be sized using a rainfall intensity of 0.3 inches per hour, which will result in treatment of the same portion of runoff as treated using volumetric standards above (criteria 3).

The preliminary sizing of the treatment control facilities is set forth in this document and the Landmark Village Drainage Concept Report (Psomas, 2006). Facility sizing will be finalized by the project engineer with the final hydrology study prior to issuance of a grading permit, which will be prepared and approved to ensure consistency with this analysis.

Also, the SUSMP includes general design specifications for individual priority project categories. These include:

- Single-Family Hillside Home
- 100,000 square foot commercial developments
- Restaurants
- Retail gasoline outlets
- Automotive repair shops
- Parking lots

For example, commercial developments must have properly designed loading and unloading dock areas, repair and maintenance bays, and vehicle equipment wash areas. Restaurants need to have properly designed equipment and accessory wash areas. Parking lots have to be properly designed to limit oil contamination and have regular maintenance of parking lot stormwater treatment systems (e.g., storm drain filters and biofilters). This document generally identifies potential locations for these types of improvements and preliminarily identifies appropriate BMPs.

The LARWQCB issued a letter in December 2006 that clarification the Board's compliance expectations for the development planning requirements in Part 4.D of the MS4 Permit (LARWQCB, 2006). Per the clarification letter, the three provisions in Part 4.D that are the essential requirements for compliance are to: (1) maximize the percentage of pervious surfaces to allow percolation of storm water into the ground; (2) minimize the quantity of storm water directed to impervious surfaces and the MS4; and (3) minimize pollution emanating from parking lots through the use of appropriate treatment control BMPs and good housekeeping practices.

The Project is required to incorporate appropriate SUSMP requirements into project plans as part of the development plan approval process for building and grading permits. This analysis will identify at a project level, and consistent with the framework, conclusions,

and requirements of the NRSP Sub-Regional Stormwater Mitigation Plan (Geosyntec Consultants, 2008), the design specifications related to treatment control BMPs and other project features associated with the Landmark Village project. Design of these BMPs will be finalized by the project engineer with the hydrology study prior to issuance of grading permits to ensure consistency with this analysis.

3.6.3 Hydromodification and Peak Flow Control

Part 4, Section D.1. of the MS4 Permit notes that increased volume, velocity, and discharge duration of stormwater runoff from developed areas may potentially accelerate downstream erosion and impair habitat-related beneficial uses in Natural Drainage Systems. As a result, Section D.1. of the Permit stipulates that Permittees shall control post-development peak storm water runoff discharge rates, velocities and durations in Natural Drainage Systems to prevent accelerated stream erosion and to protect stream habitat. Natural Drainage Systems are defined by the Permit to include the Santa Clara River.

Further, under Part 4, Section D.1 of the MS4 Permit, the County and its Co-permittees were required to develop and implement by February 1, 2005, numeric criteria for peak flow control in accordance with the findings of the Peak Discharge Impact Study analyzing the potential impacts on natural streams due to impervious development. The County of Los Angeles Department of Public Works and the Southern California Storm Water Monitoring Coalition had been conducting the study, but the study was not completed in time to meet the February 1st deadline. Therefore, on January 31, 2005, the County adopted and submitted to the LARWQCB an Interim Peak Flow Standard to be in effect until such time as a final standard can be adopted based on a completed study.

The adopted Los Angeles County Interim Peak Flow Standard was derived from a similar Interim Peak Flow Standard for Ventura County approved by the LARWQCB under the SUSMP requirements provisions of the MS4 Permit. The intent of the Interim Standard, as described by the County in the cover letter dated January 31, 2005, signed by Donald L. Wolfe transmitting the Interim Standard to Jonathan Bishop of the LARWQB, is to provide protection for natural streams to the extent supported by findings from the ongoing study, and consistent with practical construction practices.

The Interim Peak Flow Standard adopted by the County is:

The Peak Flow Standard shall require that all post-development runoff from a 2-year, 24-hour storm shall not exceed the predevelopment peak flow rate, burned, from a 2-year, 24-hour storm when the predevelopment peak flow rate equals or exceeds five cubic feet per second. Discharge flow rates shall be calculated using the County of Los Angeles Modified Rational Method. The Peak Flow Standard shall also require that post-

development runoff from the 50-year capital storm shall not exceed the predevelopment peak flow rate, burned and bulked, from the 50-year capital storm.

In its cover letter dated January 31, 2005, signed by Donald L. Wolfe, transmitting the Peak Flow Interim Standard to Jonathan Bishop of the LARWQB, the County notes that upon completion of the Peak Discharge Impact Study, new peak flow standards may be determined to be appropriate.

Per Section 4.D(9) of the MS4 Permit, the NRSP Sub-Regional Stormwater Mitigation Plan (Geosyntec Consultants, 2008) provides an alternative hydromodification control performance standard for the NRSP projects, including Landmark Village, which is sub-region specific and is based on hydrodynamic modeling and geomorphic assessment. The Landmark Village Project will be conditioned to require, as a project design feature, sizing and design of hydraulic features as necessary to control hydromodification impacts in accordance with the NSRP Sub-Regional Stormwater Mitigation Plan. See Section 5.4 below. Under Section 4.D(9) of the MS4 Permit, compliance with the NRSP Sub-Regional Stormwater Mitigation Plan hydromodification control performance standard is used to evaluate hydromodification impacts.

3.7 Construction Permits

Pursuant to the CWA Section 402(p), requiring regulations for permitting certain stormwater discharges, the State Water Resources Control Board (SWRCB) issued a statewide general NPDES Permit for stormwater discharges from construction sites [(NPDES No. CAR000002) Water Quality Order 2009-0009-DWQ, State Water Resources Control Board (SWRCB) National Pollutant Discharge Elimination System (NPDES) General Permit for Stormwater Discharges Associated with Construction Activity (adopted by the SWRCB on September 2, 2009)].

Under this Construction General Permit, discharges of stormwater from construction sites with a disturbed area of one or more acres are required to either obtain individual NPDES permits for stormwater discharges or to be covered by the Construction General Permit. Coverage under the Construction General Permit is accomplished by completing a construction site risk assessment to determine appropriate coverage level; preparing a Stormwater Pollution Prevention Plan (SWPPP), including site maps, a Construction Site Monitoring Program (CSMP), and sediment basin design calculations; for projects located outside of a Phase I or Phase II permit area, completing a post-construction water balance calculation for hydromodification controls; and completing a Notice of Intent. All of these documents must be electronically submitted to the SWRCB for General Permit coverage. The primary objective of the SWPPP is to identify and apply proper construction, implementation, and maintenance of BMPs to reduce or eliminate pollutants in stormwater discharges and authorized non-stormwater discharges from the construction site during construction. The SWPPP also outlines the monitoring and

sampling program required for the construction site to verify compliance with discharge Numeric Action Levels (NALs) set by the Construction General Permit.

3.8 General Waste Discharge Requirements for Dischargers of Groundwater From Construction and Project Dewatering

The Los Angeles Regional Water Quality Control Board has issued a General NPDES Permit and General Waste Discharge Requirements (WDRs) (Order No. R4-2008-0032, NPDES No. CAG994004) governing construction-related dewatering discharges within the Project development areas (the "General Dewatering Permit.") This permit addresses discharges from temporary dewatering operations associated with construction and permanent dewatering operations associated with development. The discharge requirements include provisions mandating notification, sampling and analysis, and reporting of dewatering and testing-related discharges. The General Dewatering Permit authorizes such construction-related activities so long as all conditions of the permit are fulfilled. Compliance with the requirements of the General Dewatering Permit is used as one method to evaluate Project construction-related impacts on surface water quality.

3.9 Discharge of Fill or Dredge Materials

Hydrologic conditions of concern addressed in this report include instream changes in sediment transport, erosion, and sedimentation, and ultimately channel stability. There is a nexus between these concerns and the stream, habitat, and species protection programs administered by the United States Army Corps of Engineers, California Department of Fish and Game and the U.S. Fish and Wildlife Service.

Section 404 of the Clean Water Act is a program that regulates the discharge of dredged and fill material into waters of the United States, including wetlands. Activities in waters of the United States that are regulated under this program include fill for development (including physical alterations to drainages to accommodate storm drainage, stabilization, and flood control improvements), water resource projects (such as dams and levees), infrastructure development (such as highways and airports), and conversion of wetlands to uplands for farming and forestry. USEPA has issued Section 404(b)(1) Guidelines (40 CFR 230) concerning the selection and use of disposal sites, including water quality aspects of such activities. Subpart C at Sections 230.20 through 230.25 contains water quality regulations applicable to dredge and fill activities. Among other topics, these guidelines address discharges that alter substrate elevation or contours, suspended particulates, water clarity, nutrients and chemical content, current patterns and water circulation, water fluctuations (including those that alter erosion or sediment rates), and salinity gradients.

Section 401 of the Clean Water Act requires that any person applying for a federal permit or license that may result in a discharge of pollutants into waters of the United States

must obtain a state water quality certification that the activity complies with all applicable water quality standards, limitations, and restrictions. Subject to certain limitations, no license or permit may be issued by a federal agency until the Section 401 certification has been granted. Further, no license or permit may be issued if certification has been denied. CWA Section 404 permits and authorizations are subject to section 401 certification by the RWQCBs.

This report does not analyze the habitat and wildlife impacts associated with physical alterations to waters of the United States proposed in conjunction with the Project, such as dredge, fill, or bed, bank or channel improvements or stabilization measures affecting waters of the U.S. The impacts associated with these physical alterations are analyzed in detail in the biota and floodplain modification sections of the Project EIR. As discussed in Section 4.4.2 below, this report analyzes the adverse impacts to natural drainage systems that may be caused by the Project's alteration of hydrologic conditions.

3.10 Lake or Streambed Alteration Agreement (LSAA)

The CDFG is responsible for conserving, protecting, and managing California's fish, wildlife, and native plant resources. To meet this responsibility, the law requires the proponent of a Project that may impact a river, stream, or lake to notify the CDFG before beginning the Project. This includes rivers or streams that flow at least periodically or permanently through a bed or channel with banks that support fish or other aquatic life and watercourses having a surface or subsurface flow that support or have supported riparian vegetation.

Section 1602 of the Fish and Game Code requires any person who proposes a Project that will substantially divert or obstruct the natural flow or substantially change the bed, channel, or bank of any river, stream, or lake or use materials from a streambed to notify the CDFG before beginning the Project. Similarly, under section 1602 of the Fish and Game Code, before any State or local governmental agency or public utility begins a construction Project that will: 1) divert, obstruct, or change the natural flow or the bed, channel, or bank of any river, stream, or lake; 2) use materials from a streambed; or 3) result in the disposal or deposition of debris, waste, or other material containing crumbled, flaked, or ground pavement where it can pass into any river, stream, or lake, it must first notify the CDFG of the proposed Project. If the CDFG determines that the Project may adversely affect existing fish and wildlife resources, a Lake or Streambed Alteration Agreement is required. In this case, on December 3, 2010, CDFG and the applicant entered into a Master Streambed Alteration Agreement (Notification No. 1600-2004-0016-R5).

As discussed above, this report does not analyze the habitat and wildlife impacts associated with physical alterations to waters of the United States proposed in conjunction with the Project, such as dredge, fill, or bed, bank or channel improvements

or stabilization measures affecting waters of the U.S. The impacts associated with these physical alterations are analyzed in detail in the biota and floodplain modification sections of the Project EIR. As discussed in Section 4.4.2 below, this report analyzes the adverse impacts to natural drainage systems that may be caused by the project's alteration of hydrologic conditions.

3.11 Los Angeles County Low Impact Development Ordinance and Manual

Chapter 12.84 of the Los Angeles County Municipal Code requires the use of low impact development ("LID") standards in development projects. This chapter applies to all development within the unincorporated area of the County after January 1, 2009, except for those developments that filed a complete discretionary or non-discretionary permit application with the Los Angeles County Department of Regional Planning, Public Works, or any County-controlled design control board, prior to January 1, 2009.

Chapter 12.84 requires that applicable development projects:

- Mimic undeveloped stormwater and urban runoff rates and volumes in any storm event up to and including the "50-year capital design storm event," as defined by LACDPW;
- Prevent pollutants of concern from leaving the development site in stormwater as the result of storms, up to and including a water quality design storm event; and
- Minimize hydromodification impacts to natural drainage systems.

To meet these standards, development projects that consist of five or more residential units, or nonresidential development, shall comply with the following:

- The excess volume (ΔV, defined as the post-developed runoff volume minus the pre-developed runoff volume for the 85th percentile storm event) from each lot upon which such development is occurring shall be infiltrated at the lot level, or in the alternative, the excess volume from the entire development site, including streets and public right-of-way, shall be infiltrated in sub-regional facilities. The tributary area of a sub-regional facility shall be limited to five acres, but may be exceeded with approval of the Director of LACDPW. When infiltration of all excess volume is not technically feasible, on-site storage, reuse, or other water conservation uses of the excess volume is required and shall be implemented as authorized by the Director of LACDPW.

LACDPW has developed a LID Standards Manual that outlines stormwater runoff quantity and quality control development principles, technologies, and design standards for achieving the LID Standards of Chapter 12.84. The LID Standards Manual requires that large scale residential and nonresidential development projects prioritize the

selection of BMPs to treat stormwater pollutants, reduce stormwater runoff volume, and promote groundwater infiltration and stormwater reuse in an integrated approach to protecting water quality and managing water resources. The Manual states that BMPs should be implemented in the following order of preference:

- BMPs that promote infiltration.
- BMPs that store and beneficially use stormwater runoff.
- BMPs that utilize the runoff for other water conservation uses including, but not limited to, BMPs that incorporate vegetation to promote pollutant removal and runoff volume reduction and integrate multiple uses, and BMPs that percolate runoff through engineered soil and allow it to discharge downstream slowly.

If compliance with the above LID requirements is technically infeasible, in whole or in part, the project must incorporate design features demonstrating compliance with the LID requirements to the maximum extent practicable. The LID goals of increasing groundwater recharge, enhancing water quality, and preventing degradation to downstream natural drainage courses will be considered by DPW in the determination of infeasibility.

The LID Standards Manual outlines site conditions where infiltration may not be possible:

- Locations where seasonal high groundwater is within 10 feet of the surface.
- Within 100 feet of a groundwater well used for drinking water.
- Brownfield development sites or other locations where pollutant mobilization is a documented concern.
- Locations with potential geotechnical hazards as outlined in a report prepared and stamped by a licensed geotechnical engineer.
- Locations with natural, undisturbed soil infiltration rates of less than 0.5 inches per hour that do not support infiltration-based BMPs.
- Locations where infiltration could cause adverse impacts to biological resources.
- Development projects in which the use of infiltration BMPs would conflict with local, State or Federal ordinances or building codes.
- Locations where infiltration would cause health and safety concerns.

The LID Standards Manual outlines where storage and reuse of the ΔV may not be possible:

- Projects that would not provide sufficient irrigation or (where permitted) domestic grey water demand for use of stored runoff due to limited landscaping or extensive use of low water use plant palettes in landscaped areas.
- Projects that are required to use recycled water for irrigation of landscaping.
- Development projects in which the storage and reuse of stormwater runoff would conflict with local, state or federal ordinances or building codes.
- Locations where storage facilities would cause potential geotechnical hazards as outlined in a report prepared and stamped by a licensed geotechnical engineer.
- Locations where storage facilities would cause health and safety concerns.

The LID Standards Manual also contains drainage analysis requirements for hydromodification impacts to off-site property. The LID Standards Manual provides for the following exemptions from conducting a full analysis for hydromodification impacts, although project applicants must still demonstrate that the project mitigates for hydromodification impacts to the satisfaction of the Director of Public Works:

- Projects that disturb less than one acre.
- Less than 10,000 square feet of new impervious area.
- Projects that do not increase impervious area or decrease the infiltration capacity of pervious areas compared to pre-project conditions.
- Projects that are replacement, maintenance, or repair of an existing permitted flood control facility.
- Projects within a watershed or subwatershed where a geomorphically-based watershed study has been prepared that establishes that the potential for hydromodification impacts is not present based on appropriate assessment and evaluation of relevant factors, including: runoff characteristics, soil conditions, watershed size and conditions, channel conditions, and proposed levels of development within the watershed.
- Projects that discharge directly or via a storm drain into concrete or significantly hardened channels, which in turn discharge into a sump area under tidal influence, or other receiving water that is not susceptible to hydromodification impacts.
- Projects that have hydrologic control measures that include sufficient subregional, regional, in-stream control measures, or a combination thereof such that hydromodification will not occur.

4 POLLUTANTS OF CONCERN AND SIGNIFICANCE CRITERIA

4.1 Surface Water Pollutants of Concern

4.1.1 Pollutants of Concern

Pollutants of concern, as defined in the Los Angeles County SUSMP Manual, consist of any pollutants that exhibit one or more of the following characteristics: current loadings or historic deposits of the pollutant are impacting the beneficial uses of a receiving water, elevated levels of the pollutant are found in sediments of a receiving water and/or have the potential to bioaccumulate in organisms therein, or the detectable inputs of the pollutant are at concentrations or loads considered potentially toxic to humans and/or flora and fauna. The pollutants of concern for the water quality analysis are those that are anticipated or potentially could be generated by the Project at concentrations, based on water quality data collected in Los Angeles County from land uses that are the same as those proposed by the Project, that exhibit these characteristics. Identification of the pollutants of concern also considered Basin Plan beneficial uses and water quality objectives, CTR criteria, and current Section 303(d) listings and TMDLs in the Santa Clara River, as well as pollutants that have the potential to cause toxicity or bioaccumulate in the receiving waters. Appendix A lists the pollutants of concern, the basis for their selection, and the significance criteria that will be applied for each.

The following pollutants were chosen as pollutants of concern for purposes of evaluating water quality based upon the above considerations:

***Sediments (TSS and Turbidity)*:** Excessive erosion, transport, and deposition of sediment in surface waters are a significant form of pollution resulting in major water quality problems. Sediment imbalances impair waters' designated uses. Excessive sediment can impair aquatic life by filling interstitial spaces of spawning gravels, impairing fish food sources, filling rearing pools, and reducing beneficial habitat structure in stream channels. In addition, excessive sediment can cause taste and odor problems in drinking water supplies and block water intake structures.

***Nutrients (Phosphorus and Nitrogen (Nitrate+Nitrite-N, Ammonia-N, and Total Nitrogen))*:** Nutrients of concern include the inorganic forms of nitrogen (nitrate, nitrite and ammonia) and phosphorus. Organic forms of nitrogen are associated with vegetative matter such as particulates from sticks and leaves. Inorganic forms of nitrogen include nitrate, nitrite and ammonia. Total Nitrogen (TN) is a measure of all nitrogen present, including inorganic and particulate forms. Phosphorus can be measured as total phosphorus (TP) or as dissolved phosphorus. Dissolved phosphorus is the more bioavailable form of phosphorus. TP is often composed mostly of soil-related particulate phosphorus. There are several sources of nutrients in urban areas, mainly fertilizers in

runoff from lawns, pet wastes, failing septic systems, atmospheric deposition from industry and automobile emissions, and soil erosion. Nutrient over-enrichment is especially prevalent in agricultural areas where manure and fertilizer inputs to crops significantly contribute to nitrogen and phosphorus levels in streams and other receiving waters. Eutrophication due to excessive nutrient input can lead to changes in algae, benthic, and fish communities; extreme eutrophication can cause hypoxia or anoxia, resulting in fish kills. Surface algal scum, water discoloration, and the release of toxins from sediment can also occur.

Various downstream reaches of the Santa Clara River are identified as impaired by ammonia and nitrate- plus nitrite-nitrogen. Evidence of impairment includes low diversity of benthic macroinvertebrates and observations of excessive algae growth. A source analysis found that the majority of ammonia and nitrate/nitrite loads are from point sources; primarily water reclamation plants (WRPs) (LARWQCB, 2003). Sources from municipal storm sewers are considered a minor source, but have a potential to cause significant local effects on water quality (LARWQCB, 2003). TMDLs have been developed and adopted into the Basin Plan for nitrogen compounds, including nitrate/nitrite and ammonia.

***Trace Metals (Aluminum, Copper, Lead, and Zinc)*:** The primary sources of trace metals in stormwater are typically commercially available metals used in transportation (e.g. automobiles), buildings, and infrastructure. Metals are also found in fuels, adhesives, paints, and other coatings. Copper, lead, and zinc are the most prevalent metals typically found in urban runoff. Other trace metals, such as cadmium, chromium, and mercury, are typically either not detected in urban runoff or are detected at very low levels (LACDPW, 2000). Metals are of concern because of the potential for toxic effects on aquatic life and the potential for groundwater contamination. High metal concentrations can lead to bioaccumulation in fish and shellfish and affect the beneficial uses of receiving waters.

Aluminum has been identified by the Los Angeles County Department of Public Works as a constituent of concern for the Santa Clara River based on monitoring conducted at mass emission Station S29 (see Section 2.7 above). In stormwater, the majority of aluminum is in the particulate phase. Its presence in stormwater is mainly due to aluminosilicate minerals found in soils, because stormwater particles are largely composed of eroded soils. Aluminum is a large component of soils and is the third most common element in the earth's crust. The average aluminum soil content is about eight percent (or 80,000 mg/kg) and suspended sediments in rivers have total aluminum contents of a similar order of magnitude. Aluminosilicates include a wide range of minerals with varying properties; some are formed during the laying down of the earth's crust and some by weathering processes. They are highly insoluble and unreactive, although aluminum can be extracted and solubilized to some degree under acidic conditions. The amount of aluminum extracted will mainly depend on the type and

particle size of aluminosilicates present in the soil matrix. A study by Kobayashi and Kizu (2001) showed that only eight percent of aluminum remained in waters after passing through a 0.22 micron filter, supporting the assertion that the majority of aluminum is found in the insoluble, suspended fraction. According to the USEPA, aluminum is not considered a contaminant of potential concern to fish or aquatic organisms when surrounding soil pH is greater than 5.5 or when in solution of a pH above 5.5 (USEPA 2003) because aluminum solubility and resultant toxicity has been linked to pH values below this standard. In general, Project area soils are not expected to have a pH of less than 5.5. DeClerk and Singer (2003) compared historic (1945) pH levels of agricultural soils in Southern California to 2001 conditions and found that pH levels have actually risen, from approximately 7.2 in 1945 to nearly 8.0 in 2001. As the majority of the pre-development land use consists of agriculture or open space, it is safe to assume that soil pH levels within the Project area will be, for the most part, above 5.5. In addition, pH in stormwater runoff is not expected to be below 5.5, as mean runoff concentrations in the Los Angeles County stormwater monitoring data ranged from 6.5 for mixed and single-family residential land uses to 7.0 for commercial land uses. In urban areas, aluminum building materials are a minor source of aluminum, as the metal is coated in unreactive aluminum oxide.

***Chloride*:** High levels of chloride in Santa Clara River Reaches 3, 5 and 6 are causing impairment of listed beneficial uses for agricultural irrigation. Irrigation of salt sensitive crops, such as avocados and strawberries, with water containing elevated levels of chloride can result in reduced crop yields. Chloride levels in some areas exceed water quality standards associated with groundwater recharge. Chloride TMDLs have been developed and adopted into the Basin Plan. The major sources of elevated chloride are dry-weather discharges from WRPs, contributing about 70% of the chloride load. Minor point sources are dewatering operations, and uncontrolled swimming pool and water ride discharges.

Pathogens (Bacteria, Viruses, and Protozoa) – Elevated pathogens are typically caused by the transport of domestic animal, wildlife, or human fecal wastes from the watershed. Runoff that flows over land such as urban runoff can mobilize pathogens, including bacteria and viruses. Even runoff from natural areas can contain pathogens (e.g., from wildlife). Other sources of pathogens in urban areas include pets, septic systems, and leaky sanitary sewer pipes. The presence of pathogens in runoff can impair receiving waters and contaminate drinking water sources. Elevated pathogens are typically caused by the transport of animal or human fecal wastes from the watershed. Historically fecal indicator bacteria (FIB), such as fecal coliform, have been used to indicate the presence of pathogens due to the difficulty of monitoring for pathogens directly. More recently, the scientific community has questioned the use of certain indicator organisms, as there are various confounding factors that affect the reliability of some FIB as pathogen indicators in stormwater runoff. Santa Clara River Reaches 5, 6, and 7 and the Santa Clara River

Estuary area identified as impaired by high fecal coliform counts from point and nonpoint sources. An Indicator Bacteria TMDL was approved by the LARWQCB for the Santa Clara River Estuary and Reaches 3, 5, 6, and 7 on July 8, 2010.

***Petroleum Hydrocarbons (Oil and Grease and PAHs)*:** The sources of oil, grease, and other petroleum hydrocarbons in urban areas include spillage fuels and lubricants, discharge of domestic and industrial wastes, atmospheric deposition, and runoff. Runoff can be contaminated by leachate from asphalt roads, wearing of tires, and deposition from automobile exhaust. Also, do-it-yourself auto mechanics may dump used oil and other automobile-related fluids directly into storm drains. Petroleum hydrocarbons, such as polycyclic aromatic hydrocarbons (PAHs), can bioaccumulate in aquatic organisms from contaminated water, sediments, and food and are toxic to aquatic life at low concentrations. Hydrocarbons can persist in sediments for long periods of time and result in adverse impacts on the diversity and abundance of benthic communities. Hydrocarbons can be measured as total petroleum hydrocarbons (TPH), oil and grease, or as individual groups of hydrocarbons, such as PAHs.

***Pesticides*:** Pesticides (including herbicides, insecticides and fungicides) are chemical compounds commonly used to control insects, rodents, plant diseases, and weeds. Excessive application of a pesticide in connection with agriculture cultivation or landscaping may result in runoff containing toxic levels of its active component. Pesticides may be classified as organochlorine pesticides or organophosphorus pesticides, the former being associated with persistent bioaccumulative pesticides (e.g., DDT and other legacy pesticides) which have been banned. The Santa Clara River estuary is listed as impaired for legacy pesticides. Organophosphorus pesticides include diazinon and chlorpyrifos whose uses also are being restricted by USEPA.

***Trash & Debris*:** Trash (such as paper, plastic, polystyrene packing foam, and aluminum materials) and biodegradable organic debris (such as leaves, grass cuttings, and food waste) are general waste products on the landscape that can be entrained in urban runoff. The presence of trash and debris may have a significant impact on the recreational value of a water body and aquatic habitat. Excess organic matter can create a high biochemical oxygen demand in a water body and thereby lower its water quality. Also, in areas where stagnant water exists, the presence of excess organic matter can promote septic conditions resulting in the growth of undesirable organisms and the release of odorous and hazardous compounds such as hydrogen sulfide.

Methylene Blue Activated Substances (MBAS). MBAS are related to the presence of detergents in water. Positive results may indicate the presence of wastewater or be associated with urban runoff due to commercial and/or residential vehicle washing or other outdoor washing activities. Surfactants disturb the surface tension which negatively affects insects and can also harm the gills in aquatic life.

Cyanide. Cyanide has been identified by the Los Angeles County Department of Public Works as a constituent of concern for the Santa Clara River based on monitoring conducted at mass emission Station S29 (LACDPW, 2005). Cyanide is used in electroplating, metallurgy, and mining. It is also used to make synthetic fibers, plastics, dyes, pharmaceuticals, and pesticides, including fumigants. In addition, cyanide serves as a chemical intermediate in various production processes. Natural cyanides are produced by certain bacteria, fungi, and algae, and they are present in a number of plants and foods as cyanogenic glycosides. Man-made cyanides typically enter the environment from metal finishing and organic chemical industries. Other sources include iron and steel works, municipal waste burning, cyanide-containing pesticides, road deicers, and vehicle exhaust.

***Bioaccumulation*:** Certain pollutants, such as pesticides, selenium and mercury, have a tendency to bioaccumulate. The Basin Plan and the CTR criteria set forth toxicity objectives for receiving water levels of substances that bioaccumulate in aquatic resources to prohibit concentrations of toxic substances that are harmful to human health and adversely affect beneficial uses.

4.1.2 Other Constituents

This section discusses other constituents that are listed in the Basin Plan, but for reasons explained below, are not pollutants of concern for the Project.

BOD (Biochemical Oxygen Demand) and Dissolved Oxygen. Adequate levels of dissolved oxygen are necessary to support aquatic life. High levels of oxygen demanding substances discharged to receiving waters can depress oxygen levels to levels of concern. Oxygen demanding substances are compounds that can be biologically degraded through aerobic processes. The presence of oxygen demanding substances can deplete oxygen supplies in waters and can contribute to algal growth. Nutrients in fertilizers and food wastes in trash are examples of likely oxygen demanding compounds to be present on the Project site. Other biodegradable organic materials include human and animal waste and vegetative matter. Biodegradable pollutants are largely subsumed by the nutrients and trash and debris categories above, and therefore will not be discussed as a separate category.

Chemical Constituents. Chemical constituents in excessive amounts in drinking water are harmful to human health. The Basin Plan objective for chemical constituents states: "Surface waters shall not contain concentrations of chemical constituents in amounts that adversely affect any designated beneficial use." As Santa Clara River Reach 5 is not designated with a municipal water supply designated use (see Section 2.5.1 above), chemical constituents are not a pollutant of concern for the Project.

Temperature. Increase in temperature can result in lower dissolved oxygen levels, impairing habitat and other beneficial uses of receiving waters. Discharges of wastewater can also cause unnatural and/or rapid changes in temperature of receiving waters, which can adversely affect aquatic life. Elevated temperatures are typically associated with discharges of process wastewaters or non-contact cooling waters. As the beneficial uses in the receiving waters for the Project include warm freshwater habitat to support warm water ecosystems, temperatures of stormwater runoff from the Project are not of concern.

Total Residual Chlorine. Total residual chlorine can be present in wastewater treatment plant discharges, or may be present in dry weather urban runoff from the emptying of swimming pools that have not been de-chlorinated. Chlorine is a strong oxidant and is therefore toxic to aquatic life. Municipal pools and private pools in areas served by a municipal sanitary system are required to be discharged into the sanitary system, and therefore, total residual chlorine will not be present in runoff from the Project.

Color, Taste, and Odor. The Basin Plan contains narrative objectives for color, taste, or odor that cause a nuisance or adversely affect beneficial uses. Undesirable tastes and odors in water may be a nuisance and may indicate the presence of a pollutant(s). Odor associated with water can result from decomposition of organic matter or the reduction of inorganic compounds, such as sulfate. Other potential sources of odor causing substances, such as industrial processes, will not occur as part of the Project. Color in water may arise naturally, such as from minerals, plant matter, or algae, or may be caused by industrial pollutants. As the Project will contain no industrial uses, color-, taste-, or odor-producing substances are not pollutants of concern for the Project.

Exotic Vegetation. Non-native (exotic) vegetation typically provides little habitat value and can out compete native vegetation that is more suitable habitat for aquatic and terrestrial organisms. The Basin Plan objective for exotic vegetation states: "Exotic vegetation shall not be introduced around stream courses to the extent that such growth causes nuisance or adversely affects designated beneficial uses." The potential for non-native plant species to impact natural drainages is analyzed in the Landmark Village Draft Environmental Impact Report Biota Section.

Mineral Quality: TDS, Sulfate, Boron, and SAR. Mineral quality in natural waters is largely determined by the mineral assemblage of soils and rocks near the land surface. Elevated mineral concentrations could impact beneficial uses; however, the minerals listed in the Basin Plan, except chloride and nitrogen, are not believed to be constituents of concern due to the absence of river impairments and/or, as with TDS, anticipated post-development runoff concentrations well below the Basin Plan objectives (Table 4-1). Therefore, these constituents are not considered pollutants of concern for the Project.

Table 4-1: Comparison of Mineral Basin Plan Objectives with Mean Measured Values in Los Angeles County

Mineral	Los Angeles Basin Plan Water Quality Objective for SCR Reach 5 (mg/L)	Range of Mean Concentration in Urban Runoff[1] (mg/L)
Total Dissolved Solids	1000	53 - 226
Sulfate	400	7 - 35
Boron	1.5	0.16 – 0.25
Sodium Absorption Ratio[2]	10	0.4 – 1.9

[1] Source: LACDPW, 2000. Land uses include SFR, MFR, commercial, education, transportation, light industrial, and mixed residential.

[2] Sodium adsorption ratio (SAR) predicts the degree to which irrigation water tends to enter into cation-exchange reactions in soil.

pH. The hydrogen ion activity of water (pH) is measured on a logarithmic scale, ranging from 0 to 14. While the pH of "pure" water at 25 ºC is 7.0, the pH of natural waters is usually slightly basic due to the solubility of carbon dioxide from the atmosphere. Aquatic organisms can be highly sensitive to pH. The Basin Plan objective for pH is:

"the pH of inland surface waters shall not be depressed below 6.5 or raised above 8.5 as a result of waste discharges. Ambient pH levels shall not be changed more than 0.5 units from natural conditions as a result of waste discharge."

Mean runoff concentrations in the Los Angeles County stormwater monitoring data ranged from 6.5 for mixed- and single-family residential land uses to 7.0 for commercial land use. Therefore, pH in the Santa Clara River is not expected to be affected by runoff discharges from the Project.

PCBs. PCBs are highly toxic persistent chemicals that have been historically released into the environment from industrial uses, such as transformers, but are no longer produced in the United States. Due to their persistence, PCBs can still be detected in urban runoff due to historic industrial sources of these chemicals. The Project area did not historically include PCB-producing land uses. Therefore, PCBs are not a pollutant of concern for the Project.

Radioactive Substances. Radioactive substances typically occur at very low concentrations in natural waters. Some activities such as mining or certain industrial activities (e.g., energy production, fuel reprocessing) can increase the amount of radioactive substances impairing beneficial uses. The Project will not have industrial or other activities that would be a source of any radioactive substances, and development will stabilize any naturally radioactive soils, though unlikely to be present in the Project area. Therefore, radioactive substances are not a pollutant of concern for the Project.

Toxicity. Certain pollutants in stormwater runoff have the potential to be highly toxic to aquatic organisms resulting in effects such as impaired reproduction or mortality. The Basin Plan water quality objective for toxicity is:

"All surface waters shall be maintained free of toxic substances in concentrations that are toxic to, or that produce detrimental physiological responses in human, plant, animal, or aquatic life."

Toxicity in urban runoff could be caused by ammonia, trace metals, PAHs, or pesticides. These constituents are subsumed by the pollutant of concern categories above.

4.2 Groundwater Pollutants of Concern

The Project will allow for incidental infiltration of urban runoff to groundwater after receiving treatment in the Project PDFs, as well as incidental infiltration of irrigation water. Research conducted on the effects on groundwater from stormwater infiltration by Pitt *et al.* (1994) indicate that the potential for contamination due to infiltration is dependent on a number of factors including the local hydrogeology and the chemical characteristics of the pollutants of concern.

Chemical characteristics that influence the potential for groundwater impacts include high mobility (low absorption potential), high solubility fractions, and abundance in runoff, including dry weather flows. As a class of constituents, trace metals tend to adsorb onto soil particles and are filtered out by the soils. This has been confirmed by extensive data collected beneath stormwater detention/retention ponds in Fresno (conducted as part of the Nationwide Urban Runoff Program (Brown & Caldwell, 1984)) that showed that trace metals tended to be adsorbed in the upper few feet in the bottom sediments. Bacteria are also filtered out by soils. More mobile and soluble pollutants, such as chloride and nitrate, have a greater potential for impacting groundwater through infiltration.

4.2.1 Pollutants of Concern

The pollutants of concern for the groundwater quality analysis are those that are anticipated or potentially could be generated by the Project at concentrations, based on water quality data collected in Los Angeles County from land uses that are the same as those included in the Project, that exhibit these characteristics. Identification of the pollutants of concern for the Project considered proposed land uses as well as pollutants that have the potential to impair beneficial uses of the groundwaters below the Project. The Los Angeles Basin Plan contains numerical objectives for bacteria, mineral quality, nitrogen, and various toxic chemical compounds, and contains qualitative objectives for taste and odor.

Nitrate+nitrite-N was chosen as the pollutant of concern for purposes of evaluating groundwater quality impacts based upon the above considerations. High nitrate levels in drinking water can cause health problems in humans. Infants can develop methemoglobinemia (blue-baby syndrome). Human activities and land use practices can influence nitrogen concentrations in groundwaters. For example, irrigation water containing fertilizers can increase levels of nitrogen in groundwater.

4.2.2 Other Constituents

Bacteria: The Basin Plan contains numeric criteria for bacteria in drinking water sources. As bacteria are removed through straining in soils (for example, as with septic tank discharges), incidental infiltration of runoff in the Project treatment PDFs is not expected to affect bacteria levels in groundwater. The WRP will include a disinfection process to reduce bacteria below levels of concern, and therefore bacteria in irrigation water are not expected to impact groundwater.

Chemical Constituents and Radioactivity: Drinking water limits for inorganic and organic chemicals that can be toxic to human health in excessive amounts and radionuclides are contained in Title 22 of the California Code of Regulations. These chemicals and radionuclides are not expected to occur in the Project's runoff. Title 22 specifies California's Wastewater Reclamation Criteria (WRC) and the NRSP WRP's recycled water must meet or exceed these criteria. These criteria apply to the treatment processes; treatment performance standards, such as removal efficiencies and effluent water quality; process monitoring programs, including type and frequency of monitoring; facility operation plans; and necessary reliability features. Due to compliance with these criteria, chemical constituents and radionuclides are not expected to occur in irrigation water in amounts that would impact groundwater.

Taste and Odor. The Basin Plan contains a narrative objective for taste and odor that cause a nuisance or adversely affect beneficial uses. Undesirable tastes and odors in groundwater may be a nuisance and may indicate the presence of a pollutant(s). Odor associated with water can result from natural processes, such as the decomposition of organic matter or the reduction of inorganic compounds, such as sulfate. Pollutants causing taste and odor issues are not expected to occur in stormwater or irrigation water in amounts that would impact groundwater. Other potential sources of odor causing substances, such as industrial processes, will not occur as part of the Project. Therefore, taste and odor-producing substances are not pollutants of concern for the Project.

Mineral Quality: TDS, Sulfate, Chloride, and Boron. Mineral quality in groundwaters is largely influenced by the mineral assemblage of soils and rocks that it comes into contact with. Elevated mineral concentrations could impact beneficial uses; however, the minerals listed in the Basin Plan are not believed to be pollutants of concern due to the anticipated runoff concentrations and the expected mineral concentrations in Newhall

Ranch WRP irrigation water, which are below the Basin Plan groundwater objectives (Table 4-2).

As required by the CWA, the Newhall Ranch WRP discharge permit includes effluent limitations that are protective of receiving water quality and designated beneficial uses (LARWQCB, 2007). Effluent limits in the WDR were developed based on the most stringent of applicable technology-based and water quality-based standards, including Basin Plan surface and groundwater objectives, CTR criteria, and applicable TMDL wasteload allocations. Therefore, these constituents are not considered pollutants of concern for the Project.

Table 4-2: Comparison of Basin Plan Mineral Groundwater Objectives with Mean Measured Values in Los Angeles County Urban Runoff and Anticipated Irrigation Water Quality

Mineral	Los Angeles Basin Plan Groundwater Quality Objective[1] (mg/L)	Range of Mean Concentrations in Urban Runoff[2] (mg/L)	Anticipated Average Concentration in Effluent from the NRSP WRP[3](mg/L)
Total Dissolved Solids	1,000	53 – 237	790
Sulfate	350	7 – 35	165
Chloride	150	4 – 50	<100
Boron	1.0	0.2 – 0.3	0.69

[1] Eastern Santa Clara-Castaic Valley

[2] Source: LACDPW, 2000. Includes all monitored land uses.

[3] Source: CH2M Hill, 2007.

4.3 Hydrologic Conditions of Concern (Hydromodification)

Urbanization modifies natural watershed and stream hydrologic and geomorphic processes by introducing increased volumes and duration of flow via increased runoff from impervious surfaces and drainage infrastructure. Several studies have evaluated affects of increased runoff associated with the introduction of impervious surfaces and drainage facilities on geomorphic processes (SCCWRP, 2005a; Geosyntec, 2002; Bledsoe & Watson, 2001; Booth, 1990; Hollis, 1975; Hammer, 1972). Potential changes to the hydrologic regime may include increases in runoff volumes, frequency of runoff events, long-term cumulative duration, as well as increased peak flows. Urbanization may also introduce dry weather flows where only wet weather flows existed prior to development. These changes are referred to as "hydromodification."

Hydromodification intensifies sediment transport and often leads to stream channel enlargement and loss of habitat and associated riparian species (SCCWRP, 2005a; Geosyntec, 2002; Bledsoe & Watson, 2001; MacRae, 1992; Booth, 1990). Under certain circumstances, development can also cause a reduction in the amount of sediment

supplied to the stream system, which can lead to stream channel incision and widening. These changes also have the potential to impact downstream channels and habitat integrity. A project that increases runoff due to impervious surfaces and traps sediment from upland watershed sources creates compounding effects.

A change to the Project site's hydrologic regime would be considered a condition of concern if the change could have a significant impact on downstream natural channels and habitat integrity, alone or in conjunction with impacts of other projects.

4.4 Significance Criteria and Thresholds for Significance

4.4.1 Surface Water Quality Significance Thresholds

Appendix A provides the criteria for evaluating the significance of a potential impact for each pollutant of concern. These criteria and the threshold for significance can be summarized as follows. The application of the criteria to a decision regarding significance requires an integrated or "weight of evidence" approach, rather than a decision based on any one of the individual criterion.

Thresholds of significance for surface water quality impacts have been developed based on a review of the MS4 Permit and the CEQA Guidelines, Appendix G. Significant adverse water quality impacts are presumed to occur if the proposed Project would:

- Create sizeable additional sources of polluted runoff to receiving waters that would result in exceedances of receiving water quality or substantially degrade water quality in receiving waters.
- Create sizeable additional sources of polluted runoff that would violate any water quality standards or waste discharge requirements for surface water runoff.
- Create sizeable additional sources of polluted construction site runoff (including polluted discharges associated with construction activities such as materials delivery, staging or storage, vehicle or equipment fueling, vehicle or equipment maintenance, waste handling, or hazardous materials handling or storage) that would violate any water quality standards or waste discharge requirements for surface water runoff or groundwater discharge.

This report analyzes whether sizeable additional sources of polluted runoff may result from the Project based on the results of water quality modeling and qualitative assessments that take into account water quality controls or BMPs that are considered Project Design Features (PDFs). Any increases in pollutant concentrations or loads in runoff resulting from the development of the Project site are considered an indication of a potentially significant adverse water quality impact. If loads and concentrations resulting from development are predicted to stay the same or to be reduced when compared with

existing conditions, it is concluded that the Project will not cause a significant adverse impact to the ambient water quality of the receiving waters for that pollutant.

If pollutant loads or concentrations are expected to increase, then for both the post-development and construction phases, potential impacts are assessed by evaluating compliance of the Project, including PDFs, with applicable regulatory requirements of the MS4 Permit, including SQMP and SUSMP requirements, the Construction General Permit, and the General Dewatering Permit. Further, post-development increases in pollutant loads and concentrations are evaluated by comparing the magnitude of the increase to relevant benchmarks, including receiving water TMDLs and receiving water quality objectives and criteria from the Basin Plan and CTR, as described below.

Receiving Water Benchmarks. Comparison of post-development water quality concentrations in the runoff discharge with benchmark TMDL wasteload or load allocations for MS4 discharges establishes the likelihood that runoff would result in TMDL exceedances in receiving waters or would otherwise degrade receiving water quality.

Comparison of post-development water quality concentrations in the runoff discharge with benchmark numeric and narrative receiving water quality criteria as provided in the Basin Plan and the CTR facilitates analysis of the potential for runoff to result in exceedances of receiving water quality standards, adversely affect beneficial uses, or otherwise degrade receiving waters.

Water quality criteria are considered benchmarks for comparison purposes only, as such criteria apply within receiving waters as opposed to applying directly to runoff discharges. Narrative and numeric water quality objectives contained in the Basin Plan apply to the Project's receiving waters. Water quality criteria contained in the CTR provide concentrations that are not to be exceeded in receiving waters more than once in a three year period for those waters designated with aquatic life or human health related uses. Projections of runoff water quality are compared to the acute form of the CTR criteria (as discussed above), as stormwater runoff is associated with episodic events of limited duration, whereas chronic criteria apply to 4-day exposures which do not describe typical storm events in the Project area, which last 11 hours on average. If pollutant levels in runoff are not predicted to exceed receiving water benchmarks, it is one indication that no significant impacts will result from project development.

As there is no water quality objective or criteria for total aluminum in the Basin Plan or the CTR, the national water quality criteria recommended by the USEPA will be used for comparison (USEPA, 1988).

MS4 Permit Requirements for New Development (SUSMP). Satisfaction of MS4 Permit requirements for new development, including SUSMP requirements and SQMP requirements, and satisfaction of construction-related requirements of the Construction General Permit and General Dewatering Permit establish compliance with water quality regulatory requirements applicable to stormwater runoff.

The MS4 Permit requires that the SQMP specify BMPs that will be implemented to reduce the discharge of pollutants in stormwater to the Maximum Extent Practicable. MS4 requirements are met when new development complies with the SUSMP requirements set forth in the MS4 Permit. Under the SUSMP requirements, the essential requirements for compliance are: (1) maximizing the percentage of pervious surfaces to allow percolation of storm water into the ground; (2) minimizing the quantity of stormwater directed to impervious surfaces and the MS4; and (3) minimizing pollution emanating from parking lots through the use of appropriate treatment control BMPs and good housekeeping practices. The effectiveness of stormwater treatment controls are primarily based on two factors - the amount of runoff that is captured by the controls and the selection of BMPs to address identified pollutants of concern. Selection and numerical sizing criteria for new development treatment controls are included in the MS4 Permit and the County SUSMP Manual. If the Project PDFs meet MS4 requirements, including sizing for treatment controls and other source control and site design BMPs consistent with the SUSMP requirements, it indicates that no significant impacts will occur as the result of MS4 Permit compliance.

Construction General Permit and General Dewatering Permit. The Construction General Permit requires the development and implementation of a Storm Water Pollution Prevention Plan (SWPPP) that describes erosion and sediment control BMPs as well as material management/ non-stormwater BMPs that will be used during the construction phase of development. The General Dewatering Permit addresses discharges from permanent or temporary dewatering operations associated with construction and development and includes provisions mandating notification, sampling and analysis, and reporting of dewatering and testing-related discharges. To evaluate significance of construction phase Project water quality impacts, we evaluate whether water quality control is achieved by implementation of BMPs consistent with Best Available Technology Economically Achievable and Best Conventional Pollutant Control Technology (BAT/BCT), as required by the Construction General Permit and the General Dewatering Permit.

4.4.2 Significance Thresholds for Hydrologic Conditions of Concern (Hydromodification Impacts)

Thresholds of significance for evaluating hydrologic impacts and conditions of concern have been developed based on a review of the MS4 Permit and the CEQA Guidelines,

Appendix G. Significant adverse impacts to natural drainage systems created by altered hydrologic conditions of concern are presumed to occur if the proposed Project would:

- Substantially alter the existing drainage pattern of a natural drainage, stream, or river causing substantial erosion, siltation, or channel instability in a manner that substantially adversely affects beneficial uses; or
- Substantially increase the rates, velocities, frequencies, duration and/or seasonality of flows causing channel instability and harming sensitive habitats or species in natural drainages in a manner that substantially adversely affects beneficial uses.

4.4.3 Cumulative Impacts

CEQA requires the analysis of cumulative impacts of a project when the project's incremental effects may be significant when assessed along with the effects of past projects and the effects of other current projects, and the reasonably foreseeable effects of probable future projects. The discussion of cumulative impacts must reflect the potential severity of the impacts and their likelihood of occurrence, but the discussion and analysis need not provide as great a detail as is provided for the direct effects attributable to the Project alone. This report therefore analyzes the potential for cumulative water quality impacts, cumulative groundwater quality impacts and cumulative hydrologic impacts generally in accordance with the thresholds for direct impacts discussed in Sections 4.4.1 and 4.4.2 above, and Section 4.4.4 below. See Sections 7.7, 7.8, and 7.9 below.

The cumulative analysis of all surface water quality and hydrologic impacts in this report is based primarily on "adopted plans and projections" found in the Los Angeles County Department of Public Works adopted and approved Hydrology Manual, which have been verified by reference to approved plans, including the City of Santa Clarita and County of Los Angeles adopted General Plans, as well as available empirical data for the Santa Clara River. As required by CEQA, the focus of the cumulative impacts analysis for this Project will be on the Project's incremental contribution to significant adverse water quality and hydrologic impacts to the SCR, taking into account the reasonably foreseeable water quality and hydrologic impacts of other projects that may develop impervious surfaces and urban land uses within the SCR watershed in accordance with adopted general plans and related projections. The cumulative impacts analysis will consider the Project's incremental contribution to significant cumulative water quality and hydrologic impacts to the SCR in light of the water quality and hydrology impact mitigation achieved by certain of the PDFs. The analysis will also consider whether the Project, including PDFs, and future projects will comply with specific requirements in a previously approved ordinance, plan, or mitigation program (such as the Basin Plan, the CTR, the MS4 Permit, the Construction General Permit and the General Dewatering Permit) that have been adopted for the purpose of avoiding or substantially lessening the

cumulative water quality and hydrologic impact problems within the geographic area in which the Project is located.

4.4.4 Groundwater Quality Impacts

Thresholds of significance for evaluating the hydrologic and water quality impacts of the Project on groundwater have been developed based on CEQA Appendix G thresholds. Significant adverse impacts to groundwater are presumed to occur if the proposed Project would:

- Substantially deplete groundwater supplies or interfere substantially with groundwater recharge so as to cause a net deficit in aquifer volume or a lowering of the local groundwater table.
- Through changes in surface water runoff quality and quantity (including Project treatment PDFs), and changes in groundwater recharge, result in a violation of any groundwater quality standards or waste discharge requirements or otherwise substantially degrade water quality.

Groundwater quality is addressed in Sections 7.8.1 and 7.8.2. Groundwater quality benchmarks were compared with post-development runoff water quality to establish the likelihood that runoff would result in a degradation of groundwater quality. Groundwater recharge is addressed in Section 7.8.3. The hydrologic effects of the Project on groundwater were examined by comparison of historical and present levels of the underlying aquifer to determine the impact of development on aquifer volume.

5 POST DEVELOPMENT SURFACE WATER QUALITY AND HYDROMODIFICATION CONTROL PROJECT DESIGN FEATURES

Project Design Features (PDFs) for surface water quality and hydrologic impacts include site design, source control, low impact development (LID), treatment control, and hydromodification control BMPs that will be incorporated into the Project and are considered a part of the Project for impact analysis. Effective management of wet and dry weather runoff water quality begins with limiting increases in runoff pollutants and flows at the source. Site design, source control, and LID BMPs are practices designed to minimize surface runoff and the introduction of pollutants into runoff. Treatment control BMPs are designed to remove pollutants once they have been mobilized by rainfall and runoff. Hydromodification control BMPs are designed to control increases in post-development runoff flows and/or volumes. This section describes the post-development site design, source control, LID, treatment control, and hydromodification control PDFs for the Project.

5.1 SUSMP Requirements and Project Design Features

Table 5-1 summarizes the SUSMP requirements and the corresponding proposed PDFs that will be incorporated into the Project.

Table 5-1: SUSMP Requirements and Corresponding Project Design Features

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
1. Runoff Flow Control	• Control post-development peak stormwater runoff discharge rates, velocities, and duration in Natural Drainage Systems to prevent accelerated downstream erosion and to protect habitat related beneficial uses.[2] • All post-development runoff from a 2-year, 24-hour storm shall not exceed the predevelopment peak flow rate, burned, from a 2-year, 24-hour storm when the predevelopment peak flow rate equals or exceeds five cfs. Discharge flow rates shall be calculated using the County of Los Angeles Modified Rational Method. • Post-development runoff from the 50-year capital storm shall not exceed the predevelopment peak flow rate, burned and bulked, from the 50-year capital storm. • Control peak flow discharge to provide stream channel and over bank flood protection, based on flow design criteria selected by the local agency.	• Hydromodification source controls include minimizing impervious surfaces through clustering development and using parcel-based LID BMPs, regional LID BMPs, and single family hydrologic source controls (HSCs) to disconnect impervious surfaces and reduce runoff volumes through evapotranspiration and infiltration. • 50-year capital storm peak flow rate analysis is contained in the "Landmark Village Tentative Tract Map 53108 Drainage Concept", prepared by Psomas (Psomas, 2009)

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
2. Conserve Natural Areas	• Concentrate or cluster development on portions of a site while leaving the remaining land in a natural undisturbed condition • Limit clearing and grading of native vegetation at a site to the minimum amount needed to build lots, allow access, and provide fire protection • Maximize trees and other vegetation at each site, planting additional vegetation, clustering tree areas, and promoting the use of native and/or drought tolerant plants • Promote natural vegetation by using parking lot islands and other landscaped areas • Preserve riparian areas and wetlands	• The NRSP clusters development into villages, including Landmark Village. Approximately 74% (10,145 acres) of the NRSP subregion will remain undeveloped. • Approximately 71.3 acres (24%) of the 292.6 gross acre Landmark Village project tract map area would remain as trails, parks, vegetated slopes, open space, and water quality treatment BMPs. Additional landscaped areas would be provided in conjunction with the residential and commercial uses, resulting in approximately 36% of the tract map site being pervious. • Existing site land use is agriculture, so little or no native vegetation is found in pre-development conditions. • Site clearing and grading will be limited as necessary to allow development, allow access, and provide fire protection. • Native and/or non-native/non-invasive vegetation will be utilized within the development. • The final project stormwater system would include the use of parcel-based LID BMPs, including, but not limited to, infiltration, bioinfiltration, and biofiltration BMPs placed in common area landscaping in commercial, multi-family residential, institutional, recreational, and park areas, roadway median strips, and parking lot islands (where applicable) and regional infiltration/biofiltration facilities incorporating natural vegetation. • Riparian buffers will be preserved along the Santa Clara River corridor by clustering development upland and away from the river.
3. Minimize Stormwater Pollutants of Concern	• Minimize, to the maximum extent practicable, the introduction of pollutants of concern that may result in significant impacts generated from site runoff of directly connected impervious areas (DCIA) to the stormwater conveyance system as approved by the building official.	• LID BMPs would be selected to address the pollutants of concern for the project. These LID BMPs include infiltration, bioinfiltration, and biofiltration BMPs implemented at the parcel-scale, media filters units implemented in right-of-ways, USEPA Green Streets practices implemented in right-of-ways, as feasible, and regional infiltration/biofiltration facilities. These BMPs are designed to minimize introduction of pollutants to the Maximum Extent Practicable (MEP).

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
		• The Project will include numerous source controls, including education programs, animal waste bag stations, street sweeping and catch basin cleaning, an Integrated Pest Management (IPM) Program for common area landscaping in commercial areas and multi-family residential areas, use of native and/or non-native/non-invasive climate appropriate vegetation, use of smart irrigation control, and installation of a car wash pad in multi-family residential areas. • An education program will be implemented that includes both the education of residents and commercial businesses regarding water quality issues. Topics will include services that could affect water quality, such as carpet cleaners and others that may not properly dispose of cleaning wastes; community car washes; and residential car washing. The education program will emphasize animal waste management, such as the importance of cleaning up after pets and not feeding pigeons, seagulls, ducks, and geese. • Landscape watering in common areas, commercial areas, multiple family residential areas, and in parks will use efficient recycled water irrigation technologies with centralized irrigation controls.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
4. Protect Slopes and Channels	Project plans must include BMPs consistent with local codes and ordinances and the SUSMP requirements to decrease the potential of slopes and/or channels from eroding and impacting stormwater runoff: • Convey runoff safely from the tops of slopes and stabilize disturbed slopes • Utilize natural drainage systems to the maximum extent practicable • Control or reduce or eliminate flow to natural drainage systems to the maximum extent practicable • Stabilize permanent channel crossings • Vegetate slopes with native or drought tolerant vegetation • Install energy dissipaters, such as riprap, at the outlets of new storm drains, culverts, conduits, or channels that enter unlined channels in accordance with applicable specifications to minimize erosion with the approval of all agencies with jurisdiction, e.g., the U.S. Army Corps of Engineers and the California Department of Fish and Game.	• There are no significant slopes or natural drainage channels within the developed portion of the Project in the post-developed condition. • Natural slopes and native vegetation on slopes adjacent to the SCR will be preserved and/or, if impacted during construction, they will be restored and enhanced. Native plants will be used in all plant palettes placed on restored slopes. • Project PDFs, parcel-based BMPs, regional LID BMPs, and Single Family HSCs, and USEPA Green Streets practices (hydrologic source controls), will reduce flows to natural channels through infiltration and evapotranspiration. • The banks of the Santa Clara River at portions of this site will be stabilized primarily using buried bank stabilization per the Newhall Ranch Resource Management and Development Plan (RMDP). After the implementation of these measures and other flow control and volume reduction PDFs, the Santa Clara River will be capable of handling the expected flow volumes, velocities, and durations with no excess erosion. For a detailed description of bank stabilization see Section 2.3.3. • All outlet points to the Santa Clara River will include energy dissipaters per the Newhall Ranch RMDP. For a detailed description of energy dissipation see Section 2.3.2.
5. Provide Storm Drain System Stenciling and Signage	• All storm drain inlets and catch basins within the Project area must be stenciled with prohibitive language and/or graphical icons to discourage illegal dumping. • Signs and prohibitive language and/or graphical icons, which prohibit illegal dumping, must be posted at public access points along channels and creeks within the Project area. • Legibility of stencils and signs must be maintained.	• All storm drain inlets and water quality inlets will be stenciled or labeled. • Signs will be posted in areas where dumping could occur. • The County, a Landscape or Local Maintenance District (LMD), Home Owners Association (HOA), or other maintenance entity will maintain stencils and signs.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
6. Properly Design Outdoor Material Storage Areas	• Where proposed Project plans include outdoor areas for storage of materials that may contribute pollutants to the storm water conveyance system measures to mitigate impacts must be included.	• Pesticides, fertilizers, paints, and other hazardous materials used for maintenance of common areas, parks, commercial areas, and multifamily residential common areas will be kept in enclosed storage areas.
7. Properly Design Trash Storage Areas	All trash containers must meet the following structural or treatment control BMP requirements: • Trash container areas must have drainage from adjoining roofs and pavement diverter around the areas. • Trash container areas must be screened or walled to prevent offsite transport of trash.	• All outdoor trash storage areas will be covered and isolated from stormwater runoff.
8. Provide Proof of Ongoing BMP Maintenance	• Applicant required to provide verification of maintenance provisions through such means as may be appropriate, including, but not limited to legal agreements, covenants, and/or Conditional Use Permits.	• Depending on the type and location of the BMP, either the County, a Landscape or Local Maintenance District (LMD), or Home Owners Association (HOA) will be responsible for maintenance of regional BMPs. The County will have the right, but not the duty, to inspect and maintain the BMPs that are maintained by the HOA or LMD, at the expense of the HOA or LMD, if they are not being properly maintained. • The HOA or commercial/business owners would be responsible for operation and maintenance of parcel-based BMPs such as bioretention placed in common area landscaping and parking lot islands. • Home owners will be responsible for maintenance of HSCs on single family parcels.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
9. Design Standards for Structural or Treatment Control BMPs	• Post-construction Structural or Treatment Control BMPs shall be designed to mitigate (infiltrate or treat) stormwater runoff using either volumetric treatment control BMPs or flow-based treatment control BMPs sized per listed criteria (see section 3.6.2 above).	• LID and treatment control BMPs will be designed to meet or exceed the sizing standards in the Los Angeles County SUSMP requirements. • Volume-based treatment control BMPs for the Project will be designed to capture 80 percent or more of the annual runoff volume per Criteria 2 of the MS4 Permit. • Flow-based treatment control BMPs will be sized using Criteria 3, which will provide 80 percent capture of annual runoff volume per criteria of the MS4 Permit. • The size of the facilities will be finalized during the design stage by the project engineer with the final hydrology study, which will be prepared and approved to ensure consistency with this analysis prior to issuance of a final grading permit. • Types of LID and treatment control BMPs that would be employed include parcel-based BMPs, regional LID BMPs, single family HSCs, USEPA Green Streets practices, media filtration, and a combination thereof.
10.B.1 Properly Design Loading/ Unloading Dock Areas (100,000 ft^2 Commercial Developments)	• Cover loading dock areas or design drainage to minimize run-on and runoff of stormwater • Direct connections to storm drains from depressed loading docks (truck wells) are prohibited	• Loading dock areas will be covered or designed to preclude run-on and runoff. • Direct connections to storm drains from depressed loading docks (truck wells) will be prohibited. • Below grade loading docks for fresh food items will drain through a Treatment Control BMP applicable to the use, such as a catch basin insert. • Loading docks will be kept in a clean and orderly condition through weekly sweeping and litter control, at a minimum and immediate cleanup of spills and broken containers without the use of water.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
10B.2. Properly Design Repair/ Maintenance Bays (100,000 ft^2 Commercial Developments)	• Repair/ maintenance bays must be indoors or designed in such a way that does not allow stormwater run-on or contact with stormwater runoff. • Design a repair/maintenance bay drainage system to capture all wash water, leaks, and spills. Connect drains to a sump for collection and disposal. Direct connection of the repair/ maintenance bays to the storm drain system is prohibited. If required by local jurisdiction, obtain an Industrial Waste Discharge Permit.	• Commercial areas will not have repair/maintenance bays or the bays will comply with design requirements.
10B.3. Properly Design Vehicle/ Equipment Wash Areas (100,000 ft^2 Commercial Developments)	• Self-contained and /or covered, equipped with a clarifier, or other pretreatment facility, and properly connected to a sanitary sewer.	• Areas for washing/steam cleaning of vehicles will be self-contained or covered with a roof or overhang; will be equipped with a wash racks and with the prior approval of the sewering agency; will be equipped with a clarifier or other pretreatment facility: and will be properly connected to a sanitary sewer.
10.C. Properly Design Equipment/ Accessory Wash Areas (Restaurants)	• Self-contained, equipped with a grease trap, and properly connected to a sanitary sewer. • If the wash area is to be located outdoors, it must be covered, paved, have secondary containment, and be connected to the sanitary sewer.	• Food preparation areas shall have either contained areas or sinks, each with sanitary sewer connections for disposal of wash waters containing kitchen and food wastes. • If located outside, the containment areas or sinks shall also be structurally covered to prevent entry of storm water. Adequate signs shall be provided and appropriately placed stating the prohibition of discharging wash water to the storm drain system.
10.D. Properly design fueling area (Retail Gasoline Outlets)	• The fuel dispensing area must be covered with an overhanging roof structure or canopy. The cover's minimum dimensions must be equal to or greater than the area within the grade break. The cover must not drain onto the fuel dispensing area and the downspouts must be routed to prevent drainage across the fueling area. • The fuel dispensing area must be paved with Portland cement concrete (or equivalent smooth impervious surface). The use of asphalt concrete shall be prohibited. • The fuel dispensing areas must have a 2% to 4% slope to prevent ponding, and must be separated from the rest of the	• Retail gasoline outlets will comply with design requirements.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
	site by a grade break that prevents run-on of urban runoff. • At a minimum, the concrete fuel dispensing area must extend 6.5 feet (2.0 meters) from the corner of each fuel dispenser, or the length at which the hose and nozzle assembly may be operated plus 1 foot (0.3 meter), whichever is less.	
10.E.1. Properly design fueling area (Automotive Repair Shops)	• See requirement 10.D. above.	• Automotive repair shop fueling areas will comply with design requirements.
10.E.2. Properly design repair/ maintenance bays (Automotive Repair Shops)	• See requirement 10.B.2 above.	• Automotive repair shop repair/maintenance bays will comply with design requirements.
10.E.3. Properly design vehicle/equipment wash areas (Automotive Repair Shops)	• Self-contained and/or covered, equipped with a clarifier, or other pretreatment facility, and properly connected to a sanitary sewer or to a permitted disposal facility.	• Automotive repair shop vehicle/equipment wash areas will comply with design requirements.
10.E.4. Properly design loading/unloading dock areas (Automotive Repair Shops)	• See requirement 10.B.1. above.	• Automotive repair shop loading/unloading dock areas will comply with design requirements.
10.F.1. Properly Design Parking Area (Parking Lots)	• Reduce impervious land coverage of parking areas • Infiltrate runoff before it reaches the storm drain system • Treat runoff before it reaches storm drain system	• Commercial, multi-family, institutional, recreational, and park parking lots would incorporate parcel-based LID BMPs located in islands to promote filtration and infiltration of runoff. • Stormwater runoff from parking lots would be directed to LID BMPs, including infiltration, bioinfiltration, and biofiltration BMPs installed at the parcel scale and regional scale, and/or media filters in compliance with the LID Performance Standard.

SUSMP Requirement[1]	Criteria/ Description	Corresponding Landmark Village PDFs
10.F.2 Properly Design to Limit Oil Contamination and Perform Maintenance (Parking Lots)	• Treat to remove oil and petroleum hydrocarbons at parking lots that are heavily used. • Ensure adequate operation and maintenance of treatment systems particularly sludge and oil removal	• See above. • Treatment of runoff in LID BMPs will be used to address oil and petroleum hydrocarbons from high-use parking lots. • The Home Owners Associations or Business Owners will be responsible for operation and maintenance of LID BMPs that serve private parking lots.
13. Limitation of Use of Infiltration BMPs	• Infiltration is limited based on design of BMP, pollutant characteristics, land use, soil conditions, and traffic. • Appropriate conditions (groundwater >10 ft from grade) must exist to utilize infiltration to treat and reduce stormwater runoff for the Project.	• Per the LARWQCB Clarification Letter (LARWQCB, 2006), generally, the common pollutants in stormwater are filtered or adsorbed by soil, and unlike hydrophobic solvents and salts, do not cause groundwater contamination. In any case, infiltration of 1-2 inches of rainfall in semi-arid areas like Southern California where there is a high rate of evapo-transpiration, presents minimal risks.

[1] SUSMP Requirements 10A (Single Family Hillside Home), 11 (Waiver), and 12 (Mitigation Funding) do not apply to the proposed Project and, therefore, are not listed in Table 5-1.

[2] This requirement is from Part 4, Section D.1 of the MS4 permit.

5.2 Site Design and LID BMPs

The purpose of site design and LID BMPs, to the extent feasible, is to mimic the pre-developed hydrologic regime. (See County of Los Angeles Low Impact Development Standards Manual, January 2009.) The primary goals of site design and LID BMPs are to maintain a landscape functionally equivalent to pre-development hydrologic conditions and to minimize the generation of pollutants of concern.

Site design and LID principles include:

Minimize Impervious Area/Maximize Permeability – Principles include preserving natural open space, reducing impervious surfaces such as roads, using more permeable paving materials, reducing street widths, using minimal disturbance techniques during development to avoid soil compaction, reducing the land coverage of buildings by building taller and narrower footprints, minimizing the use of impervious materials such as decorative concrete in landscape design, and incorporating detention or infiltration into landscape design.

Minimize Directly Connected Impervious Areas (DCIAs) – Minimizing DCIA can be achieved by directing runoff from impervious areas to vegetated areas (e.g., landscaped areas or vegetated treatment control BMPs) or to LID BMPs.

Conserve Natural Areas – Conserving and protecting native soils, vegetation, and stream corridors helps to mimic the site's natural hydrologic regime. This may be accomplished by clustering development within portions of the site to conserve as much natural open space as possible, limiting the extent of clearing and grading of native vegetation, planting additional vegetation, using native and/or non-native/non-invasive vegetation in parking lot islands and other landscape areas, and preserving and/or restoring riparian areas and wetlands.

Select Appropriate Building Materials – Use of appropriate building materials reduces the generation and discharge of pollutants of concern in runoff (and is therefore also a source control BMP).

Protect Slopes and Channels – Protecting slopes and channels reduces the potential for erosion and preserves natural sediment supply.

5.2.1 NRSP Subregion LID Performance Standard

A LID Performance Standard conceptually similar to the LID requirements in the Ventura County NPDES MS4 Permit has been developed and quantified for the Project. The LID BMP Performance Standard is illustrated in Figure 5-1 and described below:

LID project design features (PDFs) shall be selected and sized to: (1) fully retain the volume of stormwater runoff produced from a 0.75 inch storm event; and (2) reduce the percentage of Effective Impervious Area (EIA) to five percent or less of the total project area within the vesting tentative tract map and associated off-site project area. Runoff from all EIA shall be subject to treatment control measures that are selected to address the pollutants of concern and are sized to capture and treat 80 percent of the average annual runoff volume.

This LID Performance Standard will be implemented as follows:

1. Institutional, commercial, multi-family residential, recreation, and park land use parcels would implement retention or biofiltration BMPs on-site to the extent feasible. Based on an assessment of feasibility, one of three BMP strategies would be applied as outlined below:

    a. *Infiltration feasible*: If it is feasible to infiltrate all of the developed area runoff produced from the 0.75 inch design storm (i.e., soil infiltration rates are at least 0.5 inches per hour, fill depth is less than 10 feet, and no infiltration geotechnical hazards exist (such as landslides and terrace

escarpments)), infiltration BMPs would be used. Infiltration BMPs include bioretention (without an underdrain) (Figure 5-2), permeable pavement (Figure 5-3), infiltration galleries (Figure 5-4), infiltration basins or trenches (Figure 5-5), or an equivalent infiltration BMP.

b. *Bioinfiltration allowable when infiltration rates or deep fill depths are present*: If the parcel has low soil infiltration rates (i.e., the soil infiltration rate is less than 0.5 inches per hour) or the depth of fill is greater than 10 feet, but no other technical infeasibility concerns exist, bioinfiltration BMPs would be used (Figure 5-6). Bioinfiltration facilities are similar to bioretention facilities with an underdrain, but they include storage below the underdrain to maximize the volume infiltrated. These facilities would retain a portion of the runoff from the design storm, then biofilter the remaining runoff from the design storm.

c. *Infiltration is not allowable*: If infiltration is technically infeasible due to geotechnical hazards or a high ground water table, then biofiltration BMPs would be used. These BMPs would biofilter the runoff produced from the design storm from the developed area. Biofiltration BMPs include vegetated swales (Figure 5-7), filter strips (Figure 5-8), and planter boxes (Figure 5-9).

2. Runoff from roofs, patios, and walkways in single family residential parcels would be distributed over landscaped areas designed to fully retain the volume of runoff from the 0.75 inch storm event (Figure 5-10). Runoff from the remaining parcel area and that which does not infiltrate in the landscaped area would flow through the storm drain system to the regional infiltration/biofiltration facilities.

3. Runoff from roadways would be retained or biofiltered in retention or biofiltration BMPs sized to capture the design storm volume or flow, per the guidance in U.S. Environmental Protection Agency's (USEPA) Managing Wet Weather with Green Infrastructure: Green Streets.

4. No more than 5% of the total project area would be treated using conventional treatment methods that address the pollutants of concern. In this case, media filters (or equivalent BMPs that address the pollutants of concern) would be

sized to capture and treat 80% of the average annual runoff volume from the allowable EIA.

5. Regional infiltration/biofiltration facilities also would be implemented. The regional facilities would be designed to incorporate a biofilter in the bottom of the facility, which would allow for infiltration if feasible, with detention storage above the biofilter. The regional facilities would infiltrate or biofilter the design storm volume that has not been retained or biofiltered on the parcels in the area tributary to the regional facility. They also would provide extended detention treatment for the additional runoff volume required to provide 80% capture and treatment of the average annual runoff volume per the Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan treatment performance standard.

5.2.2 Consideration of Spatial Scale

Site design and LID BMP implementation for the Project occurs at different spatial scales of development. These spatial scales are listed below, from larger to smaller scale:

- Ranch scale – the Newhall Ranch Specific Plan sub-region;
- Village scale – the Landmark Village Project;
- Land use scale – single family residential, multi-family residential, commercial, education, parks, and roadways within the Landmark Village project, and
- Lot or parcel scale – individual lots or parcels within the Landmark Village project.

Table 5-2 below lists the site design and LID BMPs that will be implemented by the Project at each spatial scale.

Table 5-2: Landmark Village Site Design and LID BMPs

Spatial Scale	Corresponding Site Design and LID BMP
Ranch Scale	The Newhall Ranch Specific Plan (NRSP) clusters development into villages. Approximately 74% (10,145 acres) of the NRSP subregion will remain undeveloped Open Areas.
	A system of Open Areas will weave through the NRSP area. The Open Areas include community parks, prominent ridges, bluffs, slopes, creek beds, and utility and trail system easements, and would often function as a transition between development areas and the Special Management Areas (SMAs), which include the Santa Clara River corridor as well as the Newhall Ranch High Country. The Open Areas are designed to protect significant landforms and natural resources, and to provide an opportunity to protect natural resources from the proposed development.

Spatial Scale	Corresponding Site Design and LID BMP
	The NRSP Land Use Plan designates a total of 5,159 acres for the River Corridor and High Country Special Management Areas (SMAs). These SMAs are designed to protect the existing natural resources within Los Angeles County's Significant Ecological Areas SEA 20 and SEA 23.
	The 976-acre River Corridor SMA is designed to protect the sensitive biological resources in SEA 23. The River Corridor SMA will be dedicated to the Center for Natural Lands Management, and the Center will assume responsibility for management of this area.
	The largest land use designation of the NRSP Land Use Plan is the approximately 4,185-acre High Country SMA/SEA 20. The High Country SMA/SEA 20 is located in the southern portion of the sub-region and includes oak savannahs, high ridgelines, and various canyon drainages including Salt Creek, a regionally significant wildlife corridor that provides an important habitat link to the Santa Clara River. The High Country SMA/SEA 20 will be dedicated in fee to the Newhall Ranch Joint Powers Authority (JPA) consisting of the County of Los Angeles, the City of Santa Clarita, and the Santa Monica Mountains Conservancy, and this JPA will assume responsibility for management of this area.
	As a result of approval of the NRSP, the 1,517-acre portion of the Salt Creek watershed situated in Ventura County, which is under the ownership of Newhall Land, will be dedicated to the JPA. This dedication area is west of Newhall Ranch, and will be managed in the same manner as the High Country SMA, discussed above.
	Two conservation easements have been granted to CDFG for the purpose of conserving populations of spineflower that occur on the NRSP area.
Landmark Village Scale	Impervious areas will be minimized by incorporating landscaped areas into each village, including Landmark Village. Approximately 71.3 acres (24%) of the 292.6 gross acre Landmark Village project tract map area would remain as trails, parks, vegetated slopes, open space, and water quality treatment BMPs. Additional landscaped areas would be provided in conjunction with the residential and commercial uses, resulting in approximately 36% of the tract map site being pervious.
	The Landmark Village stormwater treatment system would provide treatment control for 97% of post-development impervious surface via the use of parcel-based and sub-regional LID BMPs that provide for volume reduction through infiltration and evapotranspiration. See Figure 5-1 and Tables 5-3 through 5-5.
	In areas not subject to mass grading, the smallest site disturbance area possible will be delineated and flagged; temporary storage of construction equipment will be restricted in these areas to minimize soil compaction on site. Site clearing and grading will be limited to the footprint necessary to allow development, access, and provide fire protection.
	The Santa Clara River Corridor and Chiquita Canyon, Long Canyon and Castaic Creek will be largely preserved, and development impacts to these resources will be minimized. An average buffer (the distance between the existing riparian resources and the Regional River Trail) of 150 - 200 feet will be provided along the Santa Clara River corridor; additionally, commercial, residential, and mixed use development will be setback 100 feet from the Regional River Trail outside of the Santa Clara River SMA/SEA 23, which will further separate development from the Santa Clara River corridor.

Spatial Scale	Corresponding Site Design and LID BMP
	Natural slopes and native vegetation on slopes adjacent to the Santa Clara River will be restored and enhanced.
Land Use Scale	Streets, sidewalks, and parking lot aisles will be constructed to the minimum widths specified in the NRSP and in compliance with regulations for the Americans with Disabilities Act and safety requirements for fire and emergency vehicle access.
	Portions of the Santa Clara River Regional River Trail will incorporate granular materials, or other pervious materials.
	Native and/or non-native/non-invasive, climate-appropriate vegetation that requires less watering and chemical application will be utilized within the common area landscaping in commercial areas and multi-family residential areas.
	Impervious surfaces will be minimized in common area landscape design for commercial areas and multi-family residential areas.
	Landscape watering in common areas, commercial areas, multiple family residential areas, and parks will use efficient recycled water irrigation technologies with centralized irrigation controls. Efficient irrigation for common area irrigation systems will include a combination of the following techniques: • Low volume irrigation systems will be used, including low volume sprinkler heads, drip emitters, and bubbler emitters, to minimize water use. • "Smart" irrigation controllers will be installed to control the amount of time irrigation systems are operated each day. These may include satellite controlled sensors or other equally effective technology.
Lot Scale	Parcel-based LID BMPs (e.g., bioretention areas, porous pavement, swales) would manage runoff from commercial, multi-family residential, institutional, recreational, and park land uses and infiltrate, bioinfiltration, or biofilter this runoff, as feasible based on geotechnical conditions. These BMPs would be located in parking lot islands and other on-site landscaped areas.
	Home builders would be required to implement hydrologic source controls for rooftops, patios, and walkways to retain the LID design storm volume. Hydrologic source controls include but are not limited to directing rooftop runoff through landscaped areas, installing percolation trenches, and installing rain barrels.

5.3 LID and Treatment Control BMPs

An assessment of infiltration feasibility was conducted to estimate, for the tract map area, which one of three on-site LID BMP strategies could be applied and whether proposed sub-regional bioinfiltration/biofiltration facilities would allow for infiltration. The tract map area was analyzed by the Project Geotechnical Consultant (Alan E. Seward, 2010) using geologic information, soils information, proposed remedial grading plans, final grades, and applicable feasibility criteria from the Los Angeles County LID Standards Manual. This analysis categorized project areas into three levels of infiltration feasibility:

1. Infiltration was considered to be feasible directly from the bottom of BMPs in locations where underlying soils infiltration rates were estimated to be greater than 0.5 inches per hour and the proposed depth of compacted fill was estimated to be less than 10 feet.
2. Infiltration was considered to be feasible through the use of dry wells in locations where underlying soils infiltration rates were estimated to be greater than 0.5 inches per hour and greater than 10 feet of separation was estimated to exist from the bottom of proposed fill to the seasonally high groundwater table.
3. Infiltration was considered to be partially feasible in the remaining areas. No hazards were identified that would preclude the use of some level of infiltration.

The results of this feasibility screening are illustrated in Figure 5-2. Figure 5-3 illustrates the LID BMPs for the Project area based on the feasibility screening. Approximately 97 percent of stormwater runoff from developed areas within the Project site would be routed to LID BMPs implemented at the parcel and regional scale (Figure 5-3). The remaining three percent would be treated in media filters or an equivalent treatment control BMPs that address pollutants of concern. Catch basin inserts and equivalent source control BMP also would be used in high-use parking lots to address trash. Collectively, the LID and treatment control PDFs would treat the pollutants of concern in runoff from the developed portion of the 292.6-gross-acre Landmark Village tract map area. Long Canyon Bridge would drain to a sub-regional biofiltration facility located within the tract map site. The off-site SR-126 expansion project would provide vegetated swale biofiltration for both the new and existing untreated roadway area. The utility corridor maintenance access road and potential future trail, as well as the water tank and access roads, would drain to infiltration or bioinfiltration BMPs (vegetated swale, filter strip, porous pavement, or bioretention). These LID BMPs would be designed to receive dry weather flows, small storm flows and the initial portion of large storm flows.

The proposed LID BMP and treatment control PDFs are summarized in Tables 5-3 through 5-5, are illustrated in Figures 5-4 through 5-14, and are described below. Project-related improvements at the borrow sites would not result in the introduction of impervious surfaces or any changes in drainage or hydrology characteristics. Therefore, all water quality potential impacts of runoff discharges from the borrow sites are limited to the construction phase pollutants.

- ***Parcel-based Infiltration BMPs:*** Parcel-based infiltration BMPs include bioretention (without an underdrain) (Figure 5-4), permeable pavement (Figure 5-5), infiltration galleries(Figure 5-6), infiltration basins or trenches (Figure 5-7), or an equivalent infiltration BMP.
- ***Parcel-based Bioinfiltration BMPs:*** Parcel-based bioinfiltration BMPs include bioretention (with an elevated underdrain) (Figure 5-8), vegetated swales (with

combined retention and treatment mechanisms), and other BMPs that are designed to retain a portion of the runoff from the LID design storm, then biofilter the remaining runoff from the design storm.

- ***Parcel-based Biofiltration BMPs*:** Parcel-based biofiltration BMPs provide for pollutant removal (e.g., filtration, adsorption, nutrient uptake) by filtering stormwater through the vegetation and soils. These BMPs include bioretention with underdrains, vegetated swales (Figure 5-9), filter strips (Figure 5-10), and planter boxes (Figure 5-11). In these BMPs, pore spaces and organic material in the soils help to retain water in the form of soil moisture and to promote the adsorption of pollutants (e.g., dissolved metals and petroleum hydrocarbons) into the soil matrix. Plants utilize soil moisture and promote the drying of the soil through evapotranspiration.
- ***Single Family Hydrologic Source Controls (Single Family HSCs)***: Runoff from roofs, patios, and walkways in single family residential parcels would be disconnected over landscaped areas designed to fully retain the volume of runoff from the LID design storm (0.75 inch storm event). Single Family HSCs would provide volume reduction by routing downspouts to landscaped areas (Figure 5-12), shallow percolation trenches, rain barrels or other equivalent means of retaining the LID design storm. Storage volume would be recovered via in infiltration and evapotranspiration.
- ***Sub-Regional/Regional Infiltration/Biofiltration Facilities***: Regional infiltration/ biofiltration facilities would be designed to incorporate a biofilter in the bottom of a regional basin, which would allow for infiltration if feasible, with detention storage above the biofilter (Figure 5-13). The regional facilities would infiltrate or biofilter the LID design storm volume that has not been retained or biofiltered on the parcels in the area tributary to the regional facility and would provide extended detention treatment for the additional runoff volume required to provide 80% capture and treatment of the average annual runoff volume per the Newhall Ranch Specific Plan Sub-Regional Stormwater Mitigation Plan treatment performance standard. The design of the facility would be tailored to soil and hydrogeologic conditions in the facility location.
- ***Media Filtration:*** Media filtration is typically comprised of a vault or catch basin that houses media designed to trap particulates and remove pollutants such as dissolved metals, nutrients, and hydrocarbons (Figure 5-14). During the filtering process, the treatment system also removes surface scum and floating oil and grease. Media may be contained in cartridges or be placed directly in a media bed configuration.

Table 5-3: Parcel-based LID BMP Tributary Areas and Design Requirements

Parcel-based BMP Type	Tributary Area (acres)	Design Requirements
Category 1: Infiltration Feasible	38.9	Retention of runoff produced by design storm
Category 2: Bioinfiltration	86.3	Capture and biotreatment of runoff produced by design storm; partial infiltration
Single-Family Hydrologic Source Controls	28.1	Rooftops, patios, walkways routed to pervious areas capable of managing runoff from the 0.75 inch storm event.
Routed Directly to Sub-Regional BMPs or Treatment BMPs (Green Streets and Media Filters)[1]	123.4	See Design Requirements in Table 5-4 below

[1] This represents the areas that are not directed to parcel-based BMPs prior to treatment in sub-regional BMPs or treatment BMPs. Other areas are treated by the specified parcel-based BMP prior to treatment in the sub-regional and treatment BMPs.

Table 5-4: Sub-Regional BMP Tributary Areas and Design Requirements

Sub-Regional BMP Type	Tributary Area (acres)	Design Requirements
Sub-Regional Bioinfiltration Facilities	203.8	80% capture of average annual volume from tributary area; retention of runoff required to meet LID Performance standard (after accounting for parcel-based retention)
Sub-Regional Biofiltration Facility	56.7	80% capture of average annual volume from tributary area

Table 5-5: Treatment BMP Tributary Areas and Design Requirements

Treatment BMP Type	Tributary Area (acres)	Design Requirements
On-site Treatment (Media Filter or Equivalent)	8.0	80% capture of average annual volume from tributary area
On-site Biofilter Swales (or Equivalent)	8.2	80% capture of average annual volume from tributary area
Off-site Biofilter Swales	103.6	80% capture of average annual volume from tributary area

5.4 Hydromodification Control PDFs

Post-development flows will be directed to the Santa Clara River after treatment; no flows will be directed to tributaries to the Santa Clara River. A series of progressive hydromodification control measures will be used in the Project to prevent and control hydromodification impacts to the Santa Clara River:

- Avoid, to the extent possible, the need to mitigate for hydromodification impacts by preserving natural hydrologic conditions and protecting sensitive hydrologic features, sediment sources, and sensitive habitats.

- Minimize the effects of development through site design practices (e.g., reducing connected impervious surfaces) and implementation of stormwater volume-reducing LID BMPs (project-based hydrologic source control).

- Mitigate hydromodification impacts in-stream using geomorphically-based channel design.

5.4.1 Hydrologic Source Control

Disconnecting impervious areas from the drainage network and adjacent impervious areas is a key approach to protecting channel stability. Several hydrologic source controls will be included in the Project that will limit impervious area and disconnect imperviousness to avoid and minimize hydromodification impacts:

- Site design. Site design PDFs that help to reduce runoff volume from the Project include the clustering of development into village areas, leaving large amounts of undeveloped open space within the NRSP subregion (of which Landmark Village is a part); routing of stormwater runoff to vegetated areas and/or LID BMPs; use of native or non-native/non-invasive plants in landscaped areas; and the use of efficient irrigation systems in common area landscaped areas.
- LID BMPs. The Project's LID BMPs will also serve as hydromodification source control BMPs. Parcel-based and regional LID BMPs would provide volume reduction ranging from incidental volume reduction in biofiltration BMPs (via evaporation and infiltration) up to full volume reduction of captured water in infiltration BMPs where soil and hydrogeologic conditions permit. In addition these facilities will also receive and eliminate dry weather flows.

5.4.2 Geomorphically-Referenced Channel Design

The hydromodification management approach for the Santa Clara River incorporates "geomorphically-referenced " channel design as described in SCCWRP Technical Report 450 (SCCWRP, 2005a). The goal of this approach is to preserve the natural stream channel function to the maximum extent practicable while limiting instability in stream channel morphology.

The Project's development footprint will allow for the greatest freedom possible for "natural stream channel" activity. This includes establishing buffer zones and maintaining setbacks to allow for channel movement and adjustment to changes in energy

associated with runoff. The engineered structural elements that will be implemented where needed for Santa Clara River stability include energy dissipation and geomorphically-referenced bank stabilization, pursuant to the Newhall Ranch RMDP.

- *Energy Dissipation*. Energy dissipation at storm drain outfalls provides erosion protection in areas where discharges have the potential to cause localized stream erosion. Erosion protection will be provided at all storm drain outlets to the Santa Clara River.

- *Bank Stabilization*. The Project will include buried soil cement along the Santa Clara River and Castaic Creek adjacent to and downstream of the Project site. In total, approximately 18,243 linear feet (LF) of bank would be provided with buried soil cement protection. This would include approximately 10,643 feet fronting the tract map site and approximately 6,400 LF on the south bank downstream (west) of the Long Canyon Road Bridge. Additional buried bank stabilization would be constructed as part of the approved Newhall Ranch WRP and between The Old Road and the Santa Clara River to protect the utility corridor. The bank protection between The Old Road and the Santa Clara River was approved as part of the Santa Clara River Natural River Management Plan (NRMP).

  Most of the proposed bank protection would consist of buried soil cement to provide scour and freeboard flood control protection. Soil cement is a modern flood control technique used to protect against erosion while maintaining natural vegetation and soft banks. Soil cement will be buried below the existing banks of the Santa Clara River. Disturbed areas will then re-vegetated with native plant species, maintaining the natural habitat presently found along the River.

  Approximately 6,600 LF of Turf Reinforcement Mat (TRM) or similar bank stability protection would be provide along the southern edge of the utility corridor downstream or west of the tract map site. TRMs are designed to reinforce vegetation at the root and stem allowing vegetation to be used as erosion control in areas where flow conditions exceed the ability of natural vegetation to remain rooted. This includes applications with high slopes or stream banks where grouted rip-rap and concrete channels are aesthetically undesirable.

5.5 Operation and Maintenance

Depending on the type and location of the BMP, either the County, a Landscape Maintenance District (LMD), Geologic Hazard Abatement District (GHAD), Home Owners Association (HOA), or other similar government or quasi-government agency

will be responsible for maintenance of regional BMPs. LMD(s), GHAD(s), or other similar government or quasi-government agency would be formed prior to turnover of stormwater facilities, prior to the first home sale. The HOA or commercial/business owners would be responsible for operation and maintenance of parcel-based BMPs such as bioretention placed in common area landscaping or parking lot islands. Homeowners would be responsible for maintenance of HSCs on single family residential properties.

Maintenance and inspection agreements will be established as the stormwater facilities are approved and built. HOA maintenance agreements will incorporate a list of HOA responsibilities. The LMD(s), GHAD(s), or other similar government or quasi-government agency will have a mechanism and staffing to monitor, maintain, and enforce BMP maintenance. The County will have the right to inspect and maintain the BMPs that are maintained by the HOA, LMD, GHAD, or other similar agency at the expense of the HOA, LMD, GHAD, or other similar agency, if they are not being properly maintained.

Operation and maintenance activities will be conducted in compliance with maintenance requirements established in the Los Angeles County Stormwater BMP Design and Maintenance Manual.

6 WATER QUALITY ANALYSIS APPROACH

6.1 Model Description

A load-based water quality model was used to estimate pollutant loads and concentrations in Project area stormwater runoff for certain pollutants of concern for pre-development conditions and post-development conditions with the LID BMPs and treatment control measures described above. This model was coupled with hydrologic and hydraulic modules of USEPA SWMM v4.4h to quantify the volume reduction and capture efficiency of the BMPs.

Table 6-1 below provides a list of model inputs and the sources for these inputs. For further detail, see Appendix B and Appendix G to this report.

Table 6-1: Water Quality Model Input Requirements and Assumptions

Model Input	Assumption/Source
Hourly long-term rainfall record	National Climatic Data Center (NCDC) Newhall (046162) and San Fernando (047762) rain gauge data from 1969-2008
Green-Ampt soil parameters	Natural Resource Conservation Service Soil Data Mart Table 5.5.5 – Handbook of Hydrology (Maidment, ed. 2003)
Land use-based imperviousness	LA County Hydrology Manual (LACDPW, 2006b)
Land use-based stormwater runoff event mean concentrations	Los Angeles County 1994-2000 Integrated Receiving Water Impacts Report, 2000 Los Angeles County 2000-2001 Stormwater Monitoring Report, 2001 Ventura County Watershed Protection District As analyzed for the Los Angeles Structural BMP Prioritization and Assessment Tool (LACDPW, City of Los Angeles, and Heal the Bay, 2008)
Volume and flow-based BMP design criteria	80% Capture of Average Annual Runoff Volume (NRSP Sub-Regional SWMP (Geosyntec, 2008))
BMP selection criteria	Select and locate BMPs with a preference for infiltration. Select BMPs to infiltrate the runoff volume from the 0.75-inch design storm to the extent feasible and biofilter the remaining fraction of the 80 percent capture volume. Evaluate degree of feasibility of infiltration based on land use type, native soil infiltration rate, proposed cut and fill, depth to groundwater, presence of landslides that will remain after remedial grading, and other geotechnically- or ecologically-based constraints.
Volume reduction and LID BMPs analyzed quantitatively	Clustering (preservation of open space) Hydrologic source controls Distributed retention, bioinfiltration, and biofiltration BMPs Regional infiltration, bioinfiltration, and biofiltration facilities Media filters

<table>
<tr><th>Model Input</th><th>Assumption/Source</th></tr>
<tr><td>Volume reduction modeling parameters</td><td>Hydrologic source controls: equal ratio of disconnected of rooftops and patios to landscaped areas receiving disconnection
Onsite BMPs:
<table>
<tr><th>Feasibility Constraint Category</th><th>Design infiltration rate (in/hr)</th></tr>
<tr><td>Category 1: Retention</td><td>0.38</td></tr>
<tr><td>Category 2: Bioinfiltration</td><td>0.25</td></tr>
</table>
Sub-regional Bioinfiltration/Biofiltration Facilities:
<table>
<tr><th>Facility Type</th><th>Design infiltration rate (in/hr)</th></tr>
<tr><td>Sub-regional Bioinfiltration Facilities (multiple)</td><td>0.25</td></tr>
<tr><td>Sub-regional Biofiltration Facility with biofiltration, extended detention and incidental infiltration</td><td>0.25</td></tr>
</table></td></tr>
<tr><td>LID BMP effluent quality</td><td>ASCE/USEPA (American Society of Civil Engineers Urban Water Resources Research Council and United States Environmental Protection Agency) 2011, International Stormwater Best Management Practices Database (www.bmpdatabase.org);
(Reanalysis of expanded database conducted January 2011)</td></tr>
</table>

6.2 Pollutants Modeled

The appropriate form of data used to address water quality are flow composite storm event samples, which are a measure of the average water quality during the event. To obtain such data usually requires automatic samplers that collect data at a frequency that is proportionate to flow rate. The pollutants of concern for which there are sufficient flow composite sampling data in the Los Angeles County database are:

- Total Suspended Solids (sediment)
- Total Phosphorus
- Nitrate-Nitrogen, Nitrite-Nitrogen, Ammonia-Nitrogen, and Total Nitrogen
- Total Aluminum
- Dissolved Copper
- Total Lead
- Dissolved Zinc
- Chloride

The other pollutants of concern, such as pathogens, hydrocarbons, pesticides, and trash and debris, are not amenable to this type of sampling either because of short holding

times (e.g., pathogens), difficulties in obtaining a representative sample (e.g., hydrocarbons), or low detection levels (e.g., pesticides). These pollutants were addressed qualitatively using literature information and best professional judgment due to the lack of statistically reliable monitoring data for these pollutants (see Section 6.3 below).

6.3 Qualitative Impact Analysis

Post development stormwater runoff water quality impacts associated with the following pollutants of concern were addressed based on literature information and professional judgment because available data were not deemed sufficient for modeling:

- Turbidity
- Pathogens (Bacteria, Viruses, and Protozoa)
- Hydrocarbons (Oil and Grease, Polycyclic Aromatic Hydrocarbons)
- Pesticides
- Trash and Debris
- Methylene Blue Activated Substances (MBAS)
- Cyanide

Human pathogens are usually not directly measured in stormwater monitoring programs because of the difficulty and expense involved; rather, indicator bacteria such as fecal coliform or certain strains of *E. Coli* are measured. Unfortunately, these indicators are not very reliable measures of the presence of pathogens in stormwater, in part because stormwater tends to mobilize pollutants from many sources, some of which contain non-pathogenic bacteria. For this reason, and because holding times for bacterial samples are necessarily short, most stormwater programs do not collect flow-weighted composite samples that potentially could produce more reliable statistical estimates of concentrations. Fecal coliform or *E. Coli* are typically measured with grab samples, making it difficult to develop reliable EMCs. Total coliform and fecal bacteria (fecal coliform, fecal streptococcus, and fecal enterococci) were detected in stormwater samples tested in Los Angeles County at highly variable densities (or most probable number, MPN) ranging between several hundred to several million cells per 100 ml (LACDPW, 2000).

Hydrocarbons are difficult to measure because of laboratory interference effects and sample collection issues (hydrocarbons tend to coat sample bottles). Hydrocarbons are typically measured with single grab samples, making it difficult to develop reliable EMCs.

Pesticides in urban runoff are often at concentrations that are below detection limits for most commercial laboratories and therefore there are limited statistically reliable data available on pesticides in urban runoff. Pesticides were not detected in Los Angeles

County monitoring data for land use-based samples, except for diazinon and glyphosate which were detected in less than 15 percent and 7 percent of samples, respectively (LACDPW, 2000).

Turbidity, trash and debris, MBAS, and cyanide are not typically included in routine urban stormwater monitoring programs. Turbidity is not typically included in post-construction treatment control BMP effectiveness studies. Several studies conducted in the Los Angeles River basin have attempted to quantify trash generated from discrete areas, but the data represent relatively small areas or relatively short periods, or both. MBAS was included in the land use-based monitoring data, but not enough data is available for modeling purposes. Cyanide was not included in the Los Angeles County land use-based monitoring program.

Also addressed qualitatively are potential water quality impacts from runoff and dewatering discharges during construction (Section 7.4), potential water quality impacts due to pollutant bioaccumulation (Section 7.5), dry weather runoff water quality impacts (Section 7.6), and groundwater quality impacts (Section 7.8).

6.3.1 LID Performance Standard Retention Volume Conformance Analysis

The Project LID Performance Standard establishes minimum requirements for stormwater retention. An analysis was performed to demonstrate that the proposed LID BMP PDFs would provide retention volume equivalent to or greater than the Project LID Performance Standard retention volume. The following stepwise process was used to show conformance with the LID Performance Standard Retention Volume.

Step 1: Calculate LID Performance Standard

The retention volume required to meet the Project LID Performance Standard was calculated as described below:

- Tabulate non-jurisdictional project area (includes vesting tentative tract map and associated off-site project areas, minus the River Corridor).
- Calculate project total impervious area. This calculation was based on proposed land uses and average impervious rates for these land uses.
- Calculate project allowable impervious area. This was calculated as five percent of the total project area as described in the Project LID Performance Standard.
- Calculate the remaining impervious area as the difference between the total impervious area and the allowable impervious area

- Calculate runoff from the remaining impervious area for the first 0.75 inches of precipitation per the method described in the Ventura County Technical Guidance Manual (TGM):

    d. V (ac-ft) = Remaining Impervious Area (ac) × 0.75 inches × 0.95 runoff coefficient × (1ft/12 inches units conversion)

- The resulting retention storage volume is the LID Performance Standard retention volume that would apply to the Project. This performance standard is expressed in acre-feet.

Step 2: Calculate Retention Volume in Land Use BMPs (Onsite)

The retention volume provided in parcel-based LID BMPs was calculated based the application of the Project LID Performance Standard to the Project land uses and infiltration constraints (Figure 5-3), as follows:

- Categorize institutional, commercial, multi-family residential, recreation, and park land use parcels by the infiltration constraint category and calculate the area of each category of parcel-based LID BMPs. This analysis was conducted using spatial datasets characterizing infiltration feasibility factors, including depth to groundwater, natural, undisturbed soil infiltration rate, net depth of proposed cut and fill, and geotechnical hazards (see Appendix H). These datasets were overlain to characterize project areas based on the following criteria:
    - *Category 1 - Infiltration Feasible* – depth to seasonally high groundwater greater than 10 feet; net depth of fill less than 10 feet; natural, undisturbed infiltration rate greater than 0.5 inches per hour; and no identified geotechnical hazards related to infiltration of stormwater.
    - *Category 2 - Infiltration Partially Feasible* – depth to seasonally high groundwater greater than 10 feet and no identified geotechnical hazards related to infiltration of stormwater (any depth of fill or natural, undisturbed infiltration rate)
    - *Category 3 - Infiltration Infeasible* –depth to seasonally high groundwater less than 10 feet; geotechnical hazard identified that would preclude infiltration

- Calculate the retention volume per BMP area provided in parcel-based LID BMPs and sum the retention volume provided in all parcel-based LID BMPs.
    - *Category 1 - Infiltration Feasible – Infiltration BMPs*: Sum all volume provided in BMPs below the overflow elevation.

- *Category 2 - Infiltration Partially Feasible - Bioinfiltration BMPs*: Sum all volume provided below underdrains and retained in soil pores.
- *Category 3 - Infiltration Infeasible – Biofiltration BMPs:* Sum all volume retained in soil pores.

- Calculate acreage of single family detached land uses and calculate retention volume associated with infiltration and evapotranspiration by Single Family HSCs:
  - Estimate area of rooftops, patios, and walkways based on typical lot dimensions and setbacks.
  - Calculate volume of runoff from roofs, patios, and walkways for the first 0.75 inches of rainfall and tabulate this volume as retention storage.

Step 3: Calculate Retention Volume in Regional Infiltration/Biofiltration Facilities

Calculate the total storage volume provided in subregional infiltration/biofiltration facilities below the lowest discharge point.

- Retention volume in subregional bioinfiltration facilities was considered to include the volume of water in the rock layer below the facility underdrains and the volume of water retained in the pore space of biofiltration media.
- Retention volume in subregional biofiltration facilities was considered to include the volume retained in the pore space of biofiltration media.

Step 4: Compare Total Retention Volume Provided to LID Performance Standard

The total retention volume provided in parcel-based BMPs and regional LID BMPs (regional infiltration/biofiltration facilities) from Steps 2 and 3, respectively, was compared to the Project LID Performance Standard retention volume computed in Step 1. The Project is considered to meet the Project LID Performance Standard if the total retention storage volume provided is equal to or greater than the Project LID Performance Standard retention volume.

7 IMPACT ASSESSMENT

The modeled pollutant impact assessment is presented in Section 7.1 and the qualitative analyses of the remaining pollutants of concern follow in Section 7.2. Analyses of dry weather impacts and compliance with NPDES Permit requirements and construction-related requirements of the Construction General Permit and Dewatering General Permit follow the pollutant-by-pollutant impact assessment. Also included is a discussion of other considerations, including operation and maintenance, vector control, bioaccumulation, and hydrologic impacts. The analysis of cumulative impacts to surface water, groundwater, and hydromodification is also provided. A weight of evidence approach is employed using the various thresholds and significance criteria discussed in Section 4.4

7.1 Post Development Stormwater Runoff Impact Assessment for Modeled Pollutants of Concern

In this section, model results for each pollutant are evaluated in relation to the following significance criteria: (1) comparison of post-development versus pre-development stormwater quality concentrations and loads; (2) comparison with MS4 Permit, Construction General Permit, and General Dewatering Permit requirements for new development; and (3) evaluation in light of receiving water benchmarks. Pursuant to the third criterion, predicted runoff pollutant concentrations in the post-development condition, with runoff LID BMP and treatment control PDFs, are compared with benchmark receiving water quality criteria as provided in the Basin Plan and the CTR and TMDL wasteload allocations. The water quality criteria and wasteload allocations are considered benchmarks for comparison purposes only, since they do not apply directly to runoff from the Project, but the comparison provides useful information to evaluate potential impacts. A weight of evidence approach is employed in this analysis considering the various significance criteria.

Results from the water quality model for significance criterion 1 are reported in a series of tables, organized by constituent, showing predicted mean annual pollutant loads (lbs/yr) and mean annual concentrations. Projections are made for two conditions: (1) existing condition, and (2) developed condition with PDFs. Following the table comparing post-development and pre-development water quality loads and concentrations for each constituent is a table comparing the post-development (with PDFs) runoff quality to the benchmark water quality objectives and criteria and TMDL wasteload allocations for downstream reaches of the Santa Clara River. Water quality observed in the Santa Clara River is also included on these tables as a benchmark.

The area of the Project included in the model was limited to the developed portion of the Project. As no impervious surfaces will be added in the borrow areas and the utility

corridor and therefore there will be no change in runoff volume or pollutant loads and concentrations in the post-developed condition, these areas were not included in the model, but are included in the qualitative construction impact assessment.

7.1.1 Stormwater Runoff Volume

Table 7-1 shows the predicted changes in stormwater runoff mean annual volumes. Mean annual runoff volumes are expected to increase substantially with development. The increase can be explained by the increase in imperviousness associated with development of the site, as well as by the decrease in infiltration capacity of existing site soils associated with the compaction of site soils during construction. For modeling purposes, the existing agricultural land use was assumed to have an imperviousness of 15 percent to account for compaction by machinery and soil saturation due to irrigation. In contrast, single family residential land use is assumed to have an average imperviousness of 42 percent, multi-family residential land use is assumed to have an average imperviousness of 68 percent, and commercial land use is assumed to have an average imperviousness of 92 percent.

Project PDFs include site design, source control, LID, and treatment control BMPs in compliance with the SUSMP requirements. Most of the site design PDFs, especially the minimization of impervious area and the provision of 59.6 acres of trails, parks, and vegetated slopes and LID BMPs within the Project, reduce the impacts of the proposed development on increases in stormwater runoff volume. Based on the LID feasibility screening conducted for this analysis, the LID BMPs would reduce the average annual runoff volume by approximately 123 acre-feet per year.

Table 7-1: Predicted Average Annual Stormwater Runoff Volumes

Site Conditions	Average Annual Stormwater Runoff Volume (acre-ft)
Existing	130
Developed with PDFs	261
Change	131

Runoff volumes from the impervious areas within the off-site project components are likely to increase compared to the existing condition. The use of LID BMPs for these areas will minimize the increase in post-development runoff volumes.

7.1.2 TSS

Comparison of Pre- and Post-Project Conditions: Table 7-2 shows the predicted average annual TSS concentration and loads. TSS concentration is predicted to decrease as a result of the Project. This decrease can be attributed to higher EMCs observed in monitoring data from agricultural and open space land uses (the existing condition for the

site) compared with urban land uses (representative of post-development conditions). TSS load is also predicted to decrease with development despite increased runoff volumes.

Table 7-2: Predicted Average Annual TSS Concentration and Load

Site Conditions	Average Annual TSS Concentration (mg/L)	Average Annual TSS Load (tons/yr)
Existing	192	37
Developed with PDFs	33	12
Change	-159	-25

Comparison with Water Quality Criteria: The predicted average annual TSS concentration in stormwater runoff is compared with receiving water objectives and the range of observed concentrations in the Santa Clara River Reach 5 in Table 7-3. The predicted TSS load and concentration declines with development and is at the low end of the range of observed concentrations in Santa Clara River Reach 5.

Table 7-3: Comparison of Predicted TSS Concentrations with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Predicted Average Annual TSS Concentration (mg/L)	Los Angeles Basin Plan Water Quality Objectives	California Toxics Rule Criteria	Range of Observed[1] Concentrations in Santa Clara River Reach 5 (mg/L)	Average Wet Weather[2] Concentration at Station S29 (Days > 0.1")
33	Water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses	NA	32 – 51,200	1,060

[1] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.7).

[2] Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.

NA – not applicable

TSS concentrations in runoff from the impervious areas in the off-site project components are likely to decrease compared to the existing condition, similarly to the modeled project area. The use of LID BMPs will further reduce the post-development runoff TSS concentrations and loads. The average TSS concentration in runoff treated in biofilters (bioretention and vegetated swales) reported in the International Stormwater BMP database is 30.7 mg/L, which is below the range of observed concentrations in Santa Clara River Reach 5.

Based on the comprehensive site design, source control, LID, and treatment control strategy, and the comparison with available in-stream data and Basin Plan benchmark objectives, the TSS in stormwater runoff from the Project will not cause a nuisance or adversely affect beneficial uses in the receiving waters.

7.1.3 Total Phosphorus

Comparison of Pre- and Post-Project Conditions: Table 7-4 shows the predicted average total phosphorus (TP) concentrations and annual loads. TP concentration and load are predicted to decrease post-development. Because much of the total phosphorus load is associated with sediments and the sediment load and concentrations are predicted to decrease with development, the TP concentration and annual TP load are also predicted to decrease.

Table 7-4: Predicted Average Annual Total Phosphorus Concentration and Annual Load

Site Conditions	Average Annual Total Phosphorus Concentration (mg/L)	Average Annual Total Phosphorus Load (lbs/yr)
Existing	1.4	548
Developed with PDFs	0.3	193
Change	-1.1	-355

Comparison with Water Quality Criteria: There are no numeric objectives for total phosphorus in the Basin Plan. A narrative objective for biostimulatory substances in the Basin Plan states: "waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses." The low predicted total phosphorus concentrations in Project stormwater discharges are not expected to promote (i.e., increase) algal growth and therefore comply with the narrative objective for biostimulatory substances in the Basin Plan. As shown in Table 7-5, the predicted total phosphorus concentration is at the low end of the range of observed concentrations in Santa Clara River Reach 5.

Table 7-5: Comparison of Predicted Total Phosphorus Concentration with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Predicted Average Annual Total Phosphorus Concentration (mg/L)	Los Angeles Basin Plan Water Quality Objectives	California Toxics Rule Criteria	Range of Observed[1] Concentrations in Santa Clara River Reach 5 (mg/L)	Average Wet Weather[2] Concentration at Station S29 (Days > 0.1")
0.3	Waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses	NA	0.18 – 13.4	0.58

[1] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.7).

[2] Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.

NA – not applicable

Total phosphorus loads and concentrations in post-development runoff from the off-site project components are likely to increase in comparison to open space runoff and to decrease in comparison to runoff from agricultural areas. The use of LID BMPs will minimize any potential increase in post-development runoff total phosphorus concentrations and loads. The average total phosphorus concentration in runoff treated in biofilters reported in the International Stormwater BMP database is 0.46 mg/L, which is within the range of observed concentrations in Santa Clara River Reach 5.

Based on the comprehensive site design, source control, LID, and treatment control strategy and the comparison with available in-stream monitoring data and Basin Plan benchmark objectives, potential impacts associated with total phosphorus are predicted to be less than significant.

7.1.4 Nitrogen Compounds

Comparison of Pre- and Post-Project Conditions: The predicted average nitrate-nitrogen plus nitrite-nitrogen, ammonia, and total nitrogen concentrations and annual loads are summarized in Table 7-6, Table 7-7, and Table 7-8, respectively. Average concentrations and loads of nitrate-nitrogen plus nitrite nitrogen, ammonia-nitrogen, and total nitrogen are predicted to decrease. The decrease in nitrogen loads and concentrations can be attributed to higher nitrite-, nitrate-, and ammonia-nitrogen EMCs observed in monitoring data from agricultural land use versus urbanized land uses, along with nitrogen reductions in the treatment control PDFs.

Table 7-6: Predicted Average Annual Nitrate-N + Nitrite-N Concentration and Load

Site Conditions	Average Annual Nitrate+Nitrite-Nitrogen Concentration (mg/L)	Average Annual Nitrate+Nitrite-Nitrogen Load (lbs/yr)
Existing	3.0	1,219
Developed with PDFs	0.6	432
Change	-2.4	-787

Table 7-7: Predicted Average Annual Ammonia-N Concentration and Load

Site Conditions	Average Annual Ammonia-N Concentration (mg/L)	Average Annual Ammonia-N Load (lbs/yr)
Existing	0.6	215
Developed with PDFs	0.2	147
Change	-0.4	-68

Table 7-8: Predicted Average Annual Total Nitrogen-N Concentration and Load

Site Conditions	Average Annual Total Nitrogen Concentration (mg/L)	Average Annual Total Nitrogen Load (lbs/yr)
Existing	6	2,137
Developed with PDFs	2	1,277
Change	-4	-860

Comparison with Water Quality Criteria: Predicted nitrogen compound concentrations are compared to Basin Plan objectives and observed concentrations in Table 7-8. The average annual stormwater concentration of ammonia is predicted to be considerably less than the concentration-based wasteload allocation for Santa Clara River Reach 5 and the Basin Plan objective, and within the range of observed concentrations. Likewise, the average annual stormwater concentration of nitrate-N plus nitrite-N is predicted to be considerably less than the TMDL wasteload allocation and the Basin Plan water quality objective and within the range of observed concentrations for this reach of the Santa Clara River.

There are no numeric objectives for total nitrogen in the Basin Plan. A narrative objective for biostimulatory substances in the Basin Plan states: "waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses." The low predicted total nitrogen concentrations in project stormwater discharges will not promote (i.e., increase) aquatic growth and therefore comply with the narrative objective for biostimulatory substances in the Basin Plan. As shown in Table 7-9, the predicted total

nitrogen concentration is within the range of observed concentrations in Santa Clara River Reach 5.

Table 7-9: Comparison of Predicted Nitrogen Compound Concentrations with Water Quality Objectives, TMDLs, and Observed Concentrations in Santa Clara River Reach 5

Nutrient	Predicted Average Annual Concentration (mg/L)	Basin Plan Water Quality Objectives[1] (mg/L)	Wasteload Allocations for MS4 Discharges into the Santa Clara River Reach 5 (mg/L)	Range of Observed[2] Concentrations in Santa Clara River Reach 5 (mg/L)	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
Nitrate-N + Nitrite-N	0.6	5	6.8[4]	0.5 – 4.8	0.9
Ammonia-N	0.2	2.2[5]	1.75[6]	<0.005 – 1.1	0.20
Total Nitrogen	2	Waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses	NA	<0.04 – 46[7]	4.4

[1] There are no CTR criteria for nitrogen compounds.
[2] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.3.1).
[3] Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.
[4] 30-day average.
[5] 4-day average, ELS present, 90th percentile pH and temperature pairing observed at USGS Monitoring Station 11108500.
[6] 30-day average in Reach 5 below Valencia.
[7] Observed values for TKN (ammonia plus organic nitrogen).

Nitrate-N plus nitrite-N and ammonia-nitrogen concentrations in post-development runoff from the off-site project components are likely to decrease or remain the same compared to open space and agricultural runoff concentrations, although loads are likely to increase due to the increase in runoff volume. The use of LID BMPs will minimize any potential increases in post-development runoff nitrate-N plus nitrite-N or ammonia-nitrogen loads. The average nitrate-N plus nitrite-N concentration in runoff treated in biofilters reported in the International Stormwater BMP database is 0.5 mg/L and the average ammonia-nitrogen concentration is 0.1 mg/L, which are considerably less than the TMDL wasteload allocations and Basin Plan water quality objectives, and are within the range of observed concentrations for Santa Clara River Reach 5.

Based on the comprehensive site design, source control, LID, and treatment control strategy, and the comparison with available in-stream monitoring data and benchmark Basin Plan objectives and wasteload allocations, potential impacts associated with nitrogen compounds are predicted to be less than significant.

7.1.5 Metals

Comparison of Pre- and Post-Project Conditions: Projected loads and concentrations for the trace metals copper, lead, zinc, and aluminum are presented in Tables 7-10 through 7-13. Except for aluminum and lead, the projections are for the dissolved form of the metal, as it is the dissolved form to which the CTR criteria apply. Due to consistently low concentrations of dissolved lead in the available stormwater runoff data, it was not possible to develop reliable EMC parameters for most land uses for modeling the dissolved fraction of lead. This constituent was therefore modeled as the total recoverable metal. Copper, lead, and zinc are the most prevalent metals typically found in urban runoff. Other trace metals, such as cadmium, chromium, and mercury, are typically not detected in urban runoff or are detected at very low levels (LACDPW, 2000).

Post-development dissolved copper, total lead, dissolved zinc, and total aluminum loads and concentrations are projected to decrease compared to pre-development conditions. These results can be explained by the difference in EMC values observed in representative monitoring data from the pre-developed agriculture and open space condition and the post-developed urban condition (see Appendix B, Table B-12).

Project PDFs include site design, source control, LID, and treatment control BMPs in compliance with the SUSMP requirements. Specific site design PDFs that will be implemented to minimize increases in trace metals include directing drainage from impervious areas to vegetated areas and the selection of building material for roof gutters and downspouts that do not include copper or zinc. Source control PDFs that target metals include education for property owners, BMP maintenance, and street sweeping private streets and parking lots. The LID and treatment control BMPs will also reduce trace metals in the runoff from the proposed development. Only the effects of the LID and treatment control PDFs are reflected in the model results.

Table 7-10: Predicted Average Annual Dissolved Copper Concentration and Load

Site Conditions	Average Annual Dissolved Copper Concentration (µg/L)	Average Annual Dissolved Copper Load (lbs/yr)
Existing	28	10
Developed with PDFs	10	8
Change	-18	-2

Table 7-11: Predicted Average Total Lead Concentration and Annual Load

Site Conditions	Average Annual Total Lead Concentration (µg/L)	Average Annual Total Lead Load (lbs/yr)
Existing	12	4.5
Developed with PDFs	4	3.0
Change	-8	-1.5

Table 7-12: Predicted Average Annual Dissolved Zinc Concentration and Load

Site Conditions	Average Annual Dissolved Zinc Concentration (µg/L)	Average Annual Dissolved Zinc Load (lbs/yr)
Existing	185	63
Developed with PDFs	60	45
Change	-125	-18

Table 7-13: Predicted Average Annual Total Aluminum Concentration and Load

Site Conditions	Average Annual Total Aluminum Concentration (µg/L)	Average Annual Total Aluminum Load (lbs/yr)
Existing	1282	487
Developed with PDFs	323	231
Change	-959	-256

1 BMP effectiveness studies in the International Stormwater BMP database infrequently monitor aluminum; therefore, insufficient effluent data were available to model the removal effectiveness of treatment control BMPs for this water quality constituent. In order to estimate the reduction in total aluminum load and concentration, TSS removal was used as a surrogate.

Comparison with Water Quality Criteria: A narrative objective for toxic substances in the Basin Plan states: "all waters shall be maintained free of toxic substances in concentrations that are toxic to, or that produce detrimental physiological responses in human, plant, animal, or aquatic life."

The CTR criteria are the applicable water quality objectives for protection of aquatic life. The CTR criteria are expressed for acute and chronic (4-day average) conditions; however, only acute conditions were considered to be applicable for stormwater discharges because the duration of stormwater discharge is consistently less than 4 days. The CTR criteria are calculated on the basis of the hardness of the receiving waters. Lower hardness concentrations result in lower, more stringent CTR criteria. The minimum hardness value (250 mg/L as $CaCO_3$) observed in the Santa Clara River at the USGS Station 11108500 during wet weather was used as a conservative estimate; the mean observed hardness value was 660 mg/L as $CaCO_3$.

For aluminum, the National Ambient Water Quality Criteria (NAWQC) acute criterion (750 µg/L for a pH range of 6.5 to 9.0) was used as a benchmark, as the CTR does not include aluminum. Although the NAWQC criterion is in the form of acid soluble aluminum (USEPA, 1988), the available monitoring data are for either dissolved aluminum or total aluminum. Acid soluble aluminum (which is operationally defined as the aluminum that passes through a 0.45 µm membrane filter after the sample has been acidified to a pH between 1.5 and 2.0 with nitric acid) represents the forms of aluminum toxic to aquatic life or that can be readily converted to toxic forms under natural conditions. The acid soluble measurement does not measure forms of aluminum, such as aluminum that is occluded in minerals, clays, and or is strongly sorbed to particulate matter, that are not toxic and are not likely to become toxic under natural conditions. As acid soluble aluminum data is not available, total aluminum has been used in order to be conservative.

Comparison of the predicted runoff metal concentrations and the acute CTR criteria for dissolved copper, total lead, and dissolved zinc and the NAWQC criterion for aluminum are shown in Table 7-14, along with the range of observed concentrations in Santa Clara River Reach 5. The comparison of the post-developed with PDFs condition to the benchmark CTR and NAWQC values shows that all of the trace metal concentrations are below the benchmark water quality criteria. The predicted trace metal concentrations are within the range of observed concentrations in Santa Clara River Reach 5 except for dissolved zinc, which is slightly higher.

Table 7-14: Comparison of Predicted Trace Metal Concentrations with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Metal	Predicted Average Annual Concentration (µg/L)	California Toxics Rule Criteria[1] (µg/L)	Range of Observed[2] Concentrations in Santa Clara River Reach 5 (µg/L)	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
Dissolved Copper	10	32	3.3 – 22.6	7.3
Total Lead	4	260	0.6 – 40	18
Dissolved Zinc	60	250	3 – 37	19
Total Aluminum	323	N/A	131 – 19,650	5,500

1 Hardness = 250 mg/L, based on minimum observed value at USGS Station 11108500. Lead criteria is for total recoverable lead. There is no CTR criterion for aluminum.

2 Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.7).

3 Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.

Trace metals (aluminum, copper, lead, and zinc) concentrations in post-development runoff from the off-site project components are likely to decrease in comparison to concentrations in runoff from agricultural areas and to increase in comparison to

concentrations in runoff from open space. Trace metal loads are likely to increase due to the increase in runoff volume. The use of LID BMPs will minimize any potential increases in post-development runoff trace metal loads. The average trace metals concentrations in runoff treated in biofilters reported in the International Stormwater BMP database are: 7.8 µg/L (dissolved copper), 9.6 µg/L (total lead), and 32.6 µg/L (dissolved zinc). No performance data is available in the International Stormwater BMP database for treatment of aluminum in biofilters. Trace metal concentrations in runoff from the off-site project components are expected to be below all benchmark water quality criteria.

Based on the comprehensive site design, source control, and treatment strategy and the comparison with the instream water quality monitoring data and benchmark water quality criteria, the Project will not have significant impacts resulting from trace metals.

7.1.6 Chloride

Comparison of Pre- and Post-Project Conditions: Table 7-15 shows the predicted average annual chloride concentration and load. Due to the conversion from agricultural to urban land-uses and the associated EMCs, annual chloride concentration is predicted to decrease when compared to the existing conditions, although the average annual chloride load is predicted to increase slightly due to increased runoff volume.

Table 7-15: Predicted Average Annual Chloride Concentration and Load

Site Conditions	Average Annual Chloride Concentration (mg/L)	Average Annual Chloride Load (tons/yr)
Existing	20	3.7
Developed with PDFs	14	5.2
Change	-6	1.5

Comparison with Water Quality Criteria: The predicted chloride concentration in post-development Project runoff is compared to the Basin Plan water quality objective and the range of observed concentrations in Santa Clara River Reach 5 in Table 7-16. The predicted average annual chloride concentration in stormwater runoff from the Project area is at the low end of the range of observed concentrations for this pollutant and is well below the Santa Clara River Reach 5 Basin Plan water quality objective and the TMDL wasteload allocation for Santa Clara River Reach 5 (100 mg/L for both).

Table 7-16: Comparison of Predicted Chloride Concentrations with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5

Pollutant	Predicted Average Annual Concentration (mg/L)	SCR Reach 5 TMDL Wasteload Allocation & Basin Plan Water Quality Objective[1] (mg/L)	Range of Observed[2] Concentrations in Santa Clara River Reach 5 (mg/L)	Average Wet Weather[3] Concentration at Station S29 (Days > 0.1")
Chloride	14	100	3 - 121	43

[1] There are no CTR criteria for chloride.

[2] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.7).

[3] Average concentration observed in wet weather monitoring data at Station S29 for all storm events greater than 0.1 inches.

Chloride concentrations in post-development runoff from the off-site project components are likely to decrease or remain the same in comparison to runoff from open space and agricultural areas, although chloride loads are likely to increase due to increased runoff volumes. Similarly to the modeled areas, the average annual chloride concentration in stormwater runoff from the utility corridor and power substation are likely to be at the low end of the range of observed concentrations for chloride and well below the Santa Clara River Reach 5 Basin Plan water quality objective and the TMDL wasteload allocation for Santa Clara River Reach 5.

Based on the comprehensive site design, source control, LID, and treatment control strategy, and comparison with benchmark receiving water criteria and instream monitoring data, the Project is not expected to have significant water quality impacts resulting from chloride.

7.1.7 Assessment of Potential Project Impacts on Instream Concentrations

The potential for Project runoff to impact instream pollutant concentrations is a function of (1) the relative magnitudes of runoff volume and instream flow volume and (2) the relative magnitude of runoff concentrations and instream concentrations. The instream pollutant concentration with Project contributions can be calculated using a simple mass balance equation:

$$C_{IS} = \frac{V_O \times C_O + V_P \times C_P}{V_O + V_P}$$

Equation 1

Where:

C_{IS} = Instream Concentration with Project Runoff
V_O = Instream Volume Upstream of Project
C_O = Instream Concentration Upstream of Project
V_P = Volume of Runoff from Project Area

C_P = Concentration of Runoff from Project Area

This relationship can also be expressed as:

$$C_{IS} = \frac{L_O + L_P}{V_O + V_P}$$ Equation 2

Where:

L_O = Instream Constituent Load Upstream of Project
L_P = Constituent Load in Runoff from Project Area

Based on these relationships, two universal conditions can be identified under which a Project would not increase instream concentration:

- **Condition 1:** If the concentration of a constituent in Project runoff (C_P) is less than the concentration of the constituent instream (C_O), then discharges from the Project would result in a reduction of the instream concentration of that constituent; it would be not be possible for the Project's discharges to cause an increase in the instream concentration. Two extreme cases can be used to demonstrate this statement:
    - First, given that C_P is less than C_O, take the case where V_P is much less than V_O (e.g., the project size is small relative to the size of the watershed). In this case, the instream concentration would effectively equal C_O, although slightly less, indicating effectively no change in the instream concentration as a result of the project's discharges.
    - Given that C_P is less than C_O, take the case where V_P is much greater than V_O (the project size is very large relative to the size of the watershed). In this case, the instream concentration would effectively equal C_P, indicating that the project would reduce instream concentration because C_P is less than C_O.
- **Condition 2:** If the load of a constituent in Project runoff (L_P) decreases with development, but the volume of runoff from the Project increases (V_P), then the Project would be expected to result in a reduction of the instream concentration of that constituent regardless of instream volumes or concentrations. It would be impossible for the project to result in an increase in the instream concentration by reducing load but adding volume. In equation 2, this would effectively increase the numerator while reducing the denominator, which must cause the instream concentration to decrease.

The comparison of the post-developed with LID BMP PDFs to the instream concentrations for the Landmark Village Project (Table 7-3, Table 7-5, Table 7-9, Table

7-14, and Table 7-16) shows that all pollutant concentrations, except dissolved zinc, are predicted to be below the average wet-weather instream concentration (Condition 1). On this basis, the Project would be expected to result in a reduction in the instream concentrations of these constituents.

Based on predicted changes in loads and volumes as a result of the Project with LID (Table 7-14), the average annual load of dissolved zinc is predicted to decrease with development, while runoff volumes are predicted to increase (Condition 2). On this basis, the Project would be expected to result in a reduction in the instream concentrations of dissolved zinc.

7.2 Post Development Stormwater Impact Assessment for Pollutants and Basin Plan Criteria Addressed Without Modeling

7.2.1 Turbidity

Turbidity is a measure of suspended matter that interferes with the passage of light through the water or in which visual depth is restricted (Sawyer *et al.*, 1994). Turbidity may be caused by a wide variety of suspended materials, which range in size from colloidal to coarse dispersions, depending upon the degree of turbulence. In lakes or other waters existing under relatively quiescent conditions, most of the turbidity will be due to colloidal and extremely fine dispersions. In rivers under flood conditions, most of the turbidity will be due to relatively coarse dispersions. Erosion of clay and silt soils may contribute to in-stream turbidity (see discussion of hydromodification impacts in Section 7.9 below). Organic materials reaching rivers serve as food for bacteria, and the resulting bacterial growth and other microorganisms that feed upon the bacteria produce additional turbidity. Nutrients in runoff may stimulate the growth of algae, which also contributes to turbidity.

Discharges of turbid runoff are primarily of concern during the construction phase of development. Construction-related impacts are addressed in Section 7.4 below. The Construction Stormwater Pollution Prevention Plan must contain sediment and erosion control BMPs pursuant to the Construction General Permit, and those BMPs must effectively control erosion and discharge of sediment, along with other pollutants, per the Best Available Technology Economically Achievable and Best Conventional Pollutant Control Technology (BAT/BCT) standards[6]. Additionally, fertilizer control, non-visible

[6] BAT/BCT are Clean Water Act technology-based standards that are applicable to construction site stormwater discharges. Federal law specifies factors relating to the assessment of BAT including: age of the equipment and facilities involved; the process employed; the engineering aspects of the application of various types of control techniques; process changes; the cost of achieving effluent reduction; non-water quality environmental impacts (including energy requirements); and other factors as the Administrator deems appropriate. Clean Water Act §304(b)(2)(B). Factors relating to the assessment of BCT include: reasonableness of the relationship between the costs of attaining a reduction in effluent and the effluent

pollutant monitoring, and trash control BMPs in the SWPPP will combine to help control turbidity during the construction phase.

In the post-development condition, placement of impervious surfaces will serve to stabilize soils and to reduce the amount of erosion that may occur from the Project area during storm events, and will therefore decrease turbidity in the runoff (see also hydromodification impacts discussed in section 7.9 below). Project PDFs, including source controls (such as common area landscape management and common area litter control) and treatment control BMPs in compliance with the SUSMP requirements, will prevent or reduce the release of organic materials and nutrients (which might contribute to algal blooms) to receiving waters. As shown in Section 7.1 above, post-development nutrients in runoff are not expected to cause significant water quality impacts. Based on implementation of the Project PDFs and the construction-related controls outlined in Section 7.4, runoff discharges from the Project will not cause increases in turbidity which would result in adverse affects to beneficial uses in the receiving waters. Based on these considerations, the water quality impacts of the Project on turbidity are considered less than significant.

7.2.2 Pathogens

Pathogens are viruses, bacteria, and protozoa that can cause gastrointestinal and other illnesses in humans through body contact exposure. Identifying pathogens in water is difficult as the number of pathogens is fairly small, requiring sampling and filtering large volumes of water to obtain a reliable result. Traditionally, regulators have used fecal indicator bacteria (FIB), such as total and fecal coliform, enterococci, and *E. coli*, as indirect measures of the presence of pathogens, and by association, human illness risk. Early epidemiological studies (i.e., studies that investigate human illness occurrence versus environmental factors such as water quality) that linked swimming-associated gastrointestinal symptoms to *E. coli* or enterococci in swimming waters for sewage-dominated receiving waters led to the development of the current recreational water quality criteria (USEPA, 1986). In contrast to receiving waters subject to sanitary discharges, only a few epidemiological studies have evaluated the health effects of exposure to water bodies subject to discharges from storm drains and these studies focused on the effects of dry weather urban flows on recreational exposure (e.g., Haile et al, 1999 and Colford et al, 2005).

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reduction benefits derived; comparison of the cost and level of reduction of such pollutants from the discharge from publicly owned treatment works to the cost and level of reduction of such pollutants from a class or category of industrial sources; the age of the equipment and facilities involved; the process employed; the engineering aspects of the application of various types of control techniques; process changes; non-water quality environmental impact (including energy requirements); and other factors as the Administrator deems appropriate. Clean Water Act §304(b)(4)(B). The Administrator of USEPA has not issued regulations specifying BAT or BCT for construction site discharges.

Santa Clara River Bacteria TMDL

The LARWQCB approved a Basin Plan amendment on July 8, 2010, to incorporate a TMDL for Indicator Bacteria for the Santa Clara River Estuary and Reaches 3, 5, 6 and 7 of the Santa Clara River (Resolution #R10-006). The TMDL provides allowable exceedance day-based WLAs for MS4 dischargers for *E. coli* in Reaches 3, 5, 6 and 7, and for Fecal coliform, Enterococcus, and Total Coliform in the Santa Clara River Estuary. These WLAs are anticipated to be incorporated into the Los Angeles County MS4 Permit once the interim and final WLAs become effective, at which point they will become an enforceable permit provision.

The TMDL WLAs applicable to Reach 5 of the Santa Clara River are listed in Table 3-3. The Indicator Bacteria TMDL MS4 WLAs are applied in the form of allowable exceedance days. The TMDL implementation schedule deadlines applicable to Reach 5 are summarized in Table 7-16.

The Regional Board indicated in the TMDL implementation schedule that the Regional Board will reconsider the TMDL if, prior to four years after the effective date of the TMDL, one of the following occurs:

1) Monitoring or any voluntary local reference system studies justify a revision, or

2) USEPA publishes revised recommended bacteria criteria (expected in December 2012), or

3) The Regional Board adopts a separate Basin Plan amendment, suspending recreational uses in the Santa Clara River during high flows.

Table 7-17: Indicator Bacteria TMDL Implementation Schedule and Tasks

Deadline	Task
1 year after effective date of TMDL	Jurisdictions and agencies responsible for the MS4 WLAs must submit an in-stream bacteria water quality monitoring plan for the SCR watershed. The monitoring plan must be approved by the Executive Officer.
6 months after monitoring plan approval by Executive Officer	Monitoring of SCR Watershed must begin.
3 years after effective date of TMDL	Jurisdictions and agencies must submit a draft Implementation Plan outlining how to achieve compliance with the WLAs.
4 years after effective date of TMDL	Interim MS4 WLAs apply.
6 months after receipt of Regional Board comments on draft Implementation Plan	Jurisdictions and agencies must submit a final Implementation Plan and begin additional outfall monitoring.

Deadline	Task
11 years after effective date of TMDL	SCR Reaches 3, 5, 6 and 7 must achieve compliance with final WLAs for geometric mean objectives and allowable exceedance days for single sample objectives for dry weather.
17 years after effective date of TMDL	SCR Reaches 3, 5, 6 and 7 must achieve compliance with final WLAs for geometric mean objectives and allowable exceedance days for single sample objectives for wet weather.

Factors That Affect FIB Concentrations

There are various confounding factors that affect the reliability of FIB as pathogen indicators. One primary factor is that there are numerous natural or non-anthropogenic (or "zoonotic") sources of FIB in developed watersheds and their receiving water bodies, including birds and other wildlife, soils, and plant matter. Anthropogenic sources may include domesticated animals and pets, poorly functioning septic systems, sewer system overflows or spills, cross-connections between sewer and storm drains, and the utilization of outdoor areas or storm drains for human waste disposal by people without access to indoor sanitary facilities. All of these sources can contribute to the concentrations of FIB, but not all the sources may pose a comparable human health risk (USEPA, 2009).

A second confounding factor is that FIB can multiply in the field if the substrate, temperature, moisture, and nutrient conditions are suitable (MEC, 2004). This is one potential reason that FIB concentrations do not always correlate with pathogens. For example, in a field study conducted by Schroeder *et al.* (2002), pathogens (in the form of viruses, bacteria, or protozoa) were found to occur in 12 of 97 soil samples, but the samples that contained pathogens did not correlate with the samples containing concentrations of FIB. Numerous other researchers have reported that bacteria presence and even regrowth was observed in various substrates such as beach sands, wrack line (accumulation of kelp in the inter-tidal area of beaches), inter/sub-tidal sediments, and material deposited in storm drains (MEC, 2004). FIB monitoring in the Santa Ana River indicate that the ubiquity of sources and potential regrowth far exceed the human sources of fecal bacteria generated by the entire population in the watershed (Surbeck et al, 2008). Regrowth of bacteria downstream of a package treatment plant utilizing ultraviolet (UV) radiation to disinfect dry weather flows in Aliso Creek was considered a prime factor in the rapid rebound of FIB concentrations downstream of the plant (Andersen, 2005). Recent research also implicates storm drain biofilms as another urban source of FIB to receiving waters (Roberts and Kolb, 2009; Skinner et al, 2010)

A third confounding factor is that the persistence of FIB may differ from those of various pathogenic viruses, bacteria, and protozoa. Viruses, for instance, are small, low in number, and difficult to inactivate, while protozoa may form protective cysts that are resistant to destruction and render them dormant but capable of reactivating in the future.

Therefore, while some indicator bacteria may die off in the water column due to ultraviolet disinfection or other unfavorable environmental conditions (including predation and antagonism), pathogens occasionally may persist longer (Haile et. al., 1999). So while the previously two described factors may result in indicator bacteria resulting in false positive indications of public health risk, there may also be instances when indicator bacteria result in false negative indications.

Current Research Efforts to Improve Recreational Water Quality Criteria

Given the concern about the adequacy of the current recreational water quality criteria, the USEPA is undergoing a comprehensive evaluation and revision of their current FIB-based recreational water quality criteria, with completion scheduled for December 2012. To help initiate this effort, USEPA gathered 43 experts to identify research priorities needed to refine the existing criteria and transition to new methods (USEPA, 2007b). The experts identified seven topics for research, including "scientifically defensible for applications in a wide variety of geographical locations and water types" and "protective of individuals exposed to recreational waters impacted by all sorts of pathogen sources including animal feces, stormwater, and sewage" (Boehm et al, 2009). In a similar effort focused on inland waters, the Water Environment Research Federation (WERF) convened an expert panel to recommend a research program that would also support USEPA's intended revision of the water quality criteria (WERF, 2009). These various research efforts are ongoing and the USEPA will consider all submitted data as part of their recreational water quality criteria revision process.

Epidemiological Studies

Until recently, few epidemiological studies have tested the health effects of exposure to the receiving waters of direct and recent stormwater runoff, and these studies have found it difficult to link illness with stormwater sources. For instance, the Mission Bay epidemiological study (Colford *et al.*, 2005) found that "only skin rash and diarrhea were consistently elevated in swimmers versus non swimmers, the risk of illness was uncorrelated with levels of traditional water quality indicators, and State water quality thresholds were not predictive of swimming-related illnesses." Various other researchers, as part of USEPA's pathogen research program, are now conducting epidemiological studies nationwide at fresh and salt water beaches that receive wastewater and/or stormwater discharges. In southern California, the Southern California Coastal Water Research Project (SCCWRP) has been conducting a multi-year study of public health risks at marine beaches, with a final report that is scheduled for late 2011. Until these various studies are completed, however, there is no reliable documentation of the health effects caused by exposure to stormwater based on epidemiological studies.

Effects of Land Use and Runoff on FIB Concentrations

Dry weather, non-storm stream flows from undeveloped watersheds tend to have lower concentrations of FIB than dry weather urban flows, although water quality standard exceedances still occur. For instance, a recent study by SCCWRP which monitored 15 unimpaired natural southern California streams weekly during dry weather for a year showed that about 18% of the samples exceeded daily and monthly bacterial indicator thresholds, although concentrations from these unimpaired streams were one to two orders of magnitude lower than levels found in developed watersheds (Tiefenthaler, *et al.*, 2009). The study reported an average of the geometric means for *E. coli* in dry weather flows in each stream of 41 MPN/100 mL. In comparison, the Santa Clara River Bacteria TMDL numeric target is 235 MPN/100 mL for any single sample and 126 MPN/100 mL for the geometric mean *E. coli* density. The Santa Clara River bacteria TMDL WLAs are based on this and other SCCWRP reference stream and reference beach datasets, in acknowledgement of natural sources.

During wet weather, stormwater runoff can mobilize indicator bacteria from a number of watershed and instream sources, and, therefore, indicator bacteria concentrations tend to increase. For example, median stormwater runoff monitoring results for the open space land use category, as summarized by Stein et. al. (2007), include *E. Coli* concentrations of about 5,400 MPN/100 mL from the 2001-2005 Los Angeles River Watershed Wet Weather Study, and 7,200 MPN/100 mL from the National Stormwater Quality Database (Pitt *et al.*, 2003). Similarly, median open space land use stormwater runoff monitoring results include *E. coli* concentrations of 5,400 MPN/100 mL from the Stein *et al.* (2007) study based on two flow-weighted average results, and 500 MPN/100 mL for fecal coliform from a 1994-2000 Los Angeles County (2000) study based on 21 grab samples. The Santa Clara River Bacteria TMDL has incorporated allowable exceedance days to account for the fact that recreational criteria, strictly applied, are frequently exceeded even at natural, undeveloped streams and beaches. The interim and final allowable exceedance days for Reach 5 of the Santa Clara River for wet and dry weather are listed in Table 3-3.

Land use type and condition also affect runoff concentrations, and most studies show higher FIB concentrations in urban runoff than in open space runoff. Runoff from residential land uses from the Los Angeles River Watershed Wet Weather Study had a median *E. coli* concentration of about 6,300 MPN/100 mL and about 8,300 from the National Stormwater Quality Database (Table 5-2, Stein et. al, 2007). The median value of four flow-weighted average results from the Stein et. al. (2007) study was about 6,100 MPN/100mL for *E. coli* for the low density residential land use site. These data represent urban areas that in general do not have source and treatment controls, and therefore are not indicative of runoff from the proposed Project.

Runoff from agricultural watersheds involving horticulture and row cropping is known to similarly contain relatively high concentrations of FIB. Data from a stormwater drain serving an agricultural watershed with predominantly row crops in Ventura County showed median fecal coliform levels (approximately 7,000 MPN/100 mL) similar to that found for general urban runoff (Ventura County, 2005). Agricultural land and open space areas likely share some of the same wildlife sources, but livestock may be present as well. These data indicate that wildlife, livestock, plants and/or soils can be a very important source of pathogens and/or FIB.

Project Design Features that Address Pathogen Indicators

The primary sources of pathogen indicators from the Project development would likely be sediment, pet wastes, wildlife, and regrowth in the storm drain itself. Other sources of pathogens and pathogen indicators, such as cross connections between sanitary and storm sewers, are unlikely given modern sanitary sewer installation methods and inspection and maintenance practices.

The levels of bacteria in runoff from the Project would be reduced by source controls, LID BMPs, and treatment controls. The most effective means of controlling specific bacteria sources, such as pet and other animal wastes, is through source control, specifically education of pet owners, education regarding feeding (and therefore attracting) of waterfowl near waterbodies, and providing products and disposal containers that encourage and facilitate cleaning up after pets. These BMPs are specified as project source controls in Table 5-1.

Although there are limited data on the effectiveness of different types of stormwater treatment to manage pathogen indicators, treatment processes that help reduce pathogen indicators include sunlight (ultraviolet light) degradation, sedimentation, and filtration.

Bioretention, a stormwater treatment BMP which provides filtration through amended soils, is an example of an effective BMP for addressing FIB. The City of Austin, Texas conducted a number of studies on the effectiveness of sedimentation/filtration treatment systems for treating stormwater runoff (City of Austin, 1990; CWP, 1996). Most of the structures were designed to treat one-half inch of runoff. Data from four sand filters indicated a range of removals from 37 percent to 83 percent for fecal coliform, and 25 percent to 81 percent for fecal streptococci. Research on the use of filtration to remove bacteria also has been conducted in Florida by the Southwest Florida Water Management District (Kurz, 1999). Significant reductions in total and fecal coliform bacteria and the other indicators were observed between inflow and outflow samples for sand filtration. Percent reductions were measured using flow-weighted sampling techniques. Total coliform bacteria removals were less than 70 percent, and fecal coliform bacteria reduction varied from 65 percent to 100 percent.

Similarly, where soil conditions are conducive to infiltration, LID practices and stormwater treatment facilities that allow for infiltration can reduce runoff volume and treat FIB by infiltration, which in turn reduces FIB loads. In a literature summary, USEPA reported typical pathogen removal for infiltration facilities as 65 to 100 percent (USEPA, 1993). These types of BMPs are specified for incorporation into the Project where feasible to meet the LID design standards specified in Section 5 of this report, which are based on achieving equivalent pollutant control and hydrologic control as specified the LID Ordinance and Manual and in the MS4 Permit/ SUSMP Manual requirements for treatment of volume or flow of stormwater.

In summary, stormwater discharges from the Project could potentially exceed the Basin Plan standard for FIB and therefore impacts from FIB may be significant prior to mitigation. However, the FIB concentrations in runoff from the Project would be reduced through the implementation of source and treatment control PDFs. The Project will incorporate a number of source controls specific to managing FIB, including education of pet owners, education regarding feeding (and therefore attracting) of waterfowl near waterbodies, and providing products and disposal containers that encourage and facilitate cleaning up after pets. The Project will not include septic systems and the sewer system will be designed to current standards which minimizes the potential for leaks. The Project development, consistent with the MS4 permit requirements, includes a comprehensive set of source, site design, LID, and treatment control PDFs, including infiltration facilities and bioretention, selected to manage pollutants of concern, including pathogen indicators. Furthermore, the Project will comply with all future MS4 Permit provisions incorporating the TMDL wasteload allocations and implementation plan. With these PDFs, the Project would not result in substantial changes in pathogen or FIB concentrations in receiving waters causing a violation of the water quality standards or waste discharge requirements or otherwise substantially degrade water quality in the receiving waters. Water quality impacts related to pathogens would be reduced to less-than-significant.

7.2.3 Hydrocarbons

Various forms of hydrocarbons (oil and grease) are common constituents associated with urban runoff; however, these constituents are difficult to measure and are typically measured with grab samples, making it difficult to develop reliable EMCs for modeling. Based on this consideration, hydrocarbons were not modeled but are addressed qualitatively.

Hydrocarbons are a broad class of compounds, most of which are non-toxic. Hydrocarbons are hydrophobic (low solubility in water), have the potential to volatilize, and most forms are biodegradable. A subset of hydrocarbons, Polynuclear Aromatic Hydrocarbons (PAHs) can be toxic depending on the concentration levels, exposure history, and sensitivity of the receptor organisms. Of particular concern are those PAH compounds associated with transportation-related sources.

Although the concentration of hydrocarbons in runoff is expected to increase slightly under post-development conditions due to the increase in roadways, driveways, parking areas, and vehicle use, the PDFs are expected to prevent appreciable increases in hydrocarbon concentrations from leaving the Project site. Source control PDFs that address petroleum hydrocarbons include educational materials on used oil programs, carpooling, and public transportation alternatives to driving; BMP maintenance; and street sweeping private streets. Although vehicle emissions and leaks are the primary source of hydrocarbons in urban areas, it is anticipated that vehicles in the proposed development will in general be well maintained and newer models which will help to limit emissions and leaks. Lastly, the parking lot LID BMP PDFs will adsorb the low levels of emulsified oils in stormwater runoff, preventing discharge of hydrocarbons and visible film in the discharge or the coating of objects in the receiving water.

The majority of PAHs in stormwater adsorb to the organic carbon fraction of particulates in the runoff, including soot carbon generated from vehicle exhaust (Ribes *et al.*, 2003). For example, a stormwater runoff study by Marslek et al. (1997) found that the dissolved-phase PAHs represented less than 11 percent of the total concentration of PAHs. Consequently, the LID BMP and treatment control PDFs, which are designed to treat pollutants through settling, filtration, and infiltration, will be effective at treating PAHs.

Los Angeles County conducted PAH analyses on 27 stormwater samples from a variety of land uses in the period 1994-2000 (LACDPW, 2000). For those land uses where sufficient samples were taken and were above detection levels to estimate statistics, the mean concentrations of individual PAH compounds ranged from 0.04 to 0.83 µg/L. The reported means were less than acute toxicity criteria available from the literature (Suter and Tsao, 1996). Moreover, the Los Angeles County data do not account for any treatment, whereas the treatment in the PDFs should result in a reduction in hydrocarbon concentrations inclusive of PAHs. This makes it very unlikely that impacts will occur to the receiving water due to hydrocarbon loads or concentrations. On this basis, the effect of the Project on petroleum hydrocarbon levels in the receiving waters post-development is considered less than significant.

During the construction phase of the Project, hydrocarbons in site runoff could result from construction equipment/vehicle fueling or spills. Construction related impacts are addressed in Section 7.4 below. However, pursuant to the Construction General Permit, the Construction Stormwater Pollution Prevention Plan must include BMPs that address proper handling of petroleum products on the construction site, such as proper petroleum product storage and spill response practices, and those BMPs must effectively prevent the release of hydrocarbons to runoff per the Best Available Technology Economically Achievable and Best Conventional Pollutant Control Technology standards. PAH that are adsorbed to sediment during the construction phase would be effectively controlled via

the erosion and sediment control BMPs. For these reasons, construction-related water quality impacts related to hydrocarbons are considered less than significant.

7.2.4 Pesticides

Pesticides can be of concern where past farming practices involved the application of persistent organochlorine pesticides. Legacy pesticides Chlordane, Dieldrin, DDT, and Toxaphene are of particular concern, as TMDLs have been established for these pesticides in the Santa Clara River estuary, approximately 40 miles downstream of the Project and this reach of the river. Historical pesticides should no longer be discharged in the watershed except in association with erosion of sediments to which these pollutants may have adhered in the past. Site development involves the importation of approximately 6,000,000 cubic yards of soil from non-agricultural areas, as well as required remedial grading which will stabilize soils and prevent their transport from the Project site, actually reducing the potential for discharge of sediments to which historical pesticides may have adsorbed in pre-development conditions.

In the post-developed condition, pesticides will be applied to common landscaped areas and residential lawns and gardens. Pesticides that have been commonly found in urban streams include the organophosphate pesticides chlorpyrifos and diazinon (Katznelson and Mumley, 1997). However, only 0 to 13% of the samples in the Los Angeles County database had detectable levels of diazinon (depending on the land use) while levels of chlorpyrifos were below detection limits for all land uses in all samples taken between 1994 and 2000 (LACDPW, 2000). Other pesticides presented in the database were seldom measured above detection limits. Furthermore, these data represent flows from areas without treatment controls, unlike the proposed Project, which does incorporate LID BMP and treatment control PDFs.

Diazinon and chlorpyrifos are two pesticides of concern due to their potential toxicity in receiving waters. The USEPA has banned all indoor uses of diazinon in 2002 and stopped all sales for all outdoor non-agricultural use in 2003 (USEPA, June, 2002)[7]. With no

[7] Changes to the use of chlorpyrifos include reductions in the residue tolerances for agricultural use, phases out nearly all indoor and outdoor residential uses, and also stops non-residential uses where children may be exposed. In Orange County, residential use accounts for around 90% of total chlorpyrifos (USEPA, June 2002). Retail sales of chlorpyrifos were stopped by December 31, 2001, and structural (e.g. construction) uses will be phased out by December 31, 2005. Some continued uses will be allowed, for example public health use for fire ant eradication and mosquito control will be permitted by professionals.

Permissible uses of diazinon will also be restricted. All indoor uses are prohibited (as of 12/2002) and retailers were required to end sales for indoor use on December, 2002. All outdoor non-agricultural uses were phased out by December 31, 2004. Therefore it is likely that the USEPA agreement will eliminate most of the use of diazinon within the NRSP area. The use of diazinon for many agricultural crops has been eliminated (USEPA 2001), while some use of this chemical will continue to be permitted for some agricultural activities.

agricultural uses planned for the proposed Project, diazinon would not be used at the proposed Project site. The USEPA has also phased out most indoor and outdoor residential uses of chlorpyrifos and has stopped all non-residential uses where children may be exposed. Use of chlorpyrifos in the proposed Project area is not expected, with the possible exception of emergency fire ant eradications until such time as reasonable alternative products are available and only with appropriate application practices in accordance with the golf course and landscape pesticide management program.

Diazinon had long been one of the most commonly used pesticides on the market (SFBRWQCB, 2005) before its use was phased-out. Although the U.S. Environmental Protection Agency's actions eliminated most urban diazinon uses by the end of 2004, phasing out diazinon likely has increased post-2004 reliance on alternative pesticides and encouraged new pesticides to enter the marketplace.

The San Francisco Regional Water Quality Control Board commissioned a study, Insecticide Market Trends and Potential Water Quality Implications, to evaluate pesticide use trends as they relate to water quality. In 2003, on the basis of current and projected pesticide use and possible water quality risks, the report considered the pesticide alternatives of potential concern for water quality to be pyrethrums; parathyroid's (bifenthrin, cyfluthrin, cypermethrin, deltamethrin, esfenvalerate, and permethrin); carbaryl; malathion; and imidacloprid (SFBRWQCB, 2003). A more recent study also identified lambda cyhalothrin (a pyrethroid) and fipronil among pesticides of interest (SFEP, 2005).

The water quality risks posed by a pesticide relate to the quantity of the pesticide used, its runoff characteristics, and its relative toxicity in water and sediment. As urban diazinon applications are phased out, the use of some alternatives may inadvertently pose new water quality risks. Given what is known about alternative pesticide use trends, pyrethroids may be the alternatives that pose the greatest concerns for water quality (SFBRWQCB, 2005). Although pyrethroids tend to be toxic to Ceriodaphnia dubia test organisms at concentrations in water comparable to diazinon, pyrethroids do not dissolve well in water but instead adhere well to surfaces, including particles in the environment (SFBRWQCB, 2005). At equilibrium, pyrethroid concentrations in sediment are reported to be about 3,000 times greater than dissolved concentrations in water (SFBRWQCB, 2005). Thus, BMPs targeting reductions and removal of sediment loads will be effective to reduce and remove pyrethroids as well.

Source control measures such as education programs for owners, occupants, and employees in the proper application, storage, and disposal of pesticides are the most promising strategies for controlling the pesticides that will be used post-development.

Structural treatment controls are less practical because of the variety of pesticides and wide range of chemical properties that affect their ability to treat these compounds. However, most pesticides, including historical pesticides that may be present at the site, are relatively insoluble in water and therefore tend to adsorb to the surfaces of sediment, which will be stabilized with development, or if eroded, will be settled or filtered out of the water column in the LID BMP and treatment control PDFs.

For common area landscaping in commercial areas, multi-family residential areas, and parks, an Integrated Pest Management (IPM) Program will be incorporated. The goal of an IPM is to keep pest levels at or below threshold levels, reducing risk and damage from pest presence, while eliminating the risk from the pest control methods used. IPM programs achieve these goals through the use of low risk management options by emphasizing use of natural biological methods and the appropriate use of selective pesticides. IPM programs also incorporate environmental consideration by implementing procedures that minimize intrusion and alteration of biodiversity in ecosystems.

While pesticides are subject to degradation, they vary in how long they maintain their ability to eradicate pests. Some break down almost immediately into nontoxic byproducts, while others can remain active for longer periods of time. While pesticides that degrade rapidly are less likely to adversely affect non-targeted organisms, in some instances it may be more advantageous to apply longer-lasting pesticides if it results in fewer applications or smaller amounts of pesticide use. As part of the Integrated Pest Management program, careful consideration will be made as to the appropriate type of pesticides for use on the Project site. While pesticide use is likely to occur due to maintenance of landscaped areas, particularly in the residential portions of the development, careful selection, storage and application of these chemicals for use in common areas per the IPM Program will help prevent adverse water quality impacts from occurring. Additionally, as discussed above, removal of sediments in the LID and treatment control PDFs will also remove sediment-adsorbed pesticides.

Based on the incorporation of site design, source control, LID, and treatment control BMPs pursuant to SUSMP requirements and the use of an Integrated Pest Management Program, potential post-development impacts associated with pesticides are expected to be less than significant.

Transport of legacy pesticides adsorbed to existing site sediments may be a concern during the construction phase of development. Construction-related impacts are addressed in Section 7.4 below. The Construction Stormwater Pollution Prevention Plan must contain sediment and erosion control BMPs pursuant to the Construction General Permit, and those BMPs must effectively control erosion and the discharge of sediment along with other pollutants per the BAT/BCT standards. Based on these sediment

controls, construction-related impacts associated with pesticides are expected to be less than significant.

7.2.5 Trash and Debris

Urban development tends to generate significant amounts of trash and debris. Trash refers to any human-derived materials including paper, plastics, metals, glass and cloth. Debris is defined as any organic material transported by stormwater, including leaves, twigs, and grass clippings (DLWC, 1996). Debris can be associated with the natural condition. Trash and debris can be characterized as material retained on a 5-mm mesh screen. It contributes to the degradation of receiving waters by imposing an oxygen demand, attracting pests, disturbing physical habitats, clogging storm drains and conveyance culverts and mobilizing nutrients, pathogens, metals, and other pollutants that may be attached to the surface. Sources of trash in developed areas can be both accidental and intentional. During wet weather events, gross debris deposited on paved surfaces can be transported to storm drains, where it eventually can be discharged to receiving waters. Trash and debris can also be mobilized by wind and transported directly into waterways, imposing an oxygen demand on the water body as organic matter decomposes.

Urbanization could significantly increase trash and debris loads if left unchecked. However, the PDFs, including source control, LID BMP, and treatment control PDFs, will minimize the adverse impacts of trash and debris. Source controls such as street sweeping, public education, fines for littering, and storm drain stenciling can be effective in reducing the amount of trash and debris that is available for mobilization during wet and dry weather events. Common area litter control will include a litter patrol, covered trash receptacles, emptying of trash receptacles in a timely fashion, and noting trash violations by tenants/homeowners or businesses and reporting the violations to the owner/HOA for investigation. Catch basin inserts will be provided for high use parking lots. The PDFs will remove or prevent the release of floating materials, including solids, liquids, foam, or scum, from runoff discharges and will prevent impacts on dissolved oxygen in the receiving water due to decomposing debris. Based on these considerations, post-development trash and debris is not expected to significantly impact the receiving waters of the Project.

During the construction phase, there is potential for an increase in trash and debris loads due to lack of proper contractor good housekeeping practices at the construction site. Per the Construction General Permit, the SWPPP for the site will include BMPs for trash control (catch basin inserts, good housekeeping practices, etc.). Compliance with the Permit Requirements and inclusion of these BMPs, meeting BAT/BCT, included in the SWPPP will mitigate impacts from trash and debris to a level less than significant. See Section 7.4 below for a full discussion of Construction Related Impacts.

7.2.6 Methylene Blue Activated Substances (MBAS)

MBAS, which is related to the presence of detergents in runoff, may be incidentally associated with urban development due to commercial and/or residential vehicle washing or other outdoor washing activities. Surfactants disturb the surface tension which affects insects and can affect gills in aquatic life.

The presence of soap in Project runoff will be controlled through the source control PDFs, including a public education program on residential and charity car washing, and the provision of a car wash pad connected to sanitary sewer in the multi-family residential areas. Other sources of MBAS, such as cross connections between sanitary and storm sewers, are unlikely given modern sanitary sewer installation methods and inspection and maintenance practices. Therefore, MBAS are not expected to significantly impact the receiving waters of the Project.

7.2.7 Cyanide

The information on cyanide levels in urban stormwater is relatively sparse. The incidence of detection of cyanide in urban stormwater is relatively low, except in some special cases. In the Nationwide Urban Runoff Project (NURP), cyanide was detected in runoff from four cities out of a total of 15 cities that participated in the monitoring program (USEPA 1983). Overall, cyanide was detected in 23 percent of the urban runoff samples collected (16 out of a total of 71 samples), at concentrations ranging from 2 to 33 µg/L (Cole *et al.* 1984). Of the 71 samples, only 3 percent (i.e., 2) exceeded the freshwater acute guideline of 22 µg/L (USEPA 1983). The predominant sources of cyanides found in urban runoff samples were reported to be products of gasoline combustion and anti-caking ingredients in road salts (Cole *et al.* 1984).

A review of highway runoff (Colman 2001) suggested that deicing salts are the main source of cyanide in highway runoff. It has been estimated that approximately two million pounds of sodium ferrocyanide, which is used as an anticaking agent in road salts during the winter in the northeastern United States, are washed off from roads into streams and storm sewers (USEPA 1981; Gaffney *et al.* 1987). Information on the quality of snow packs and snow melt support the premise that deicing salts are the major source of cyanide in stormwater. For example, concentrations of cyanide in snow packs ranged up to 314 µg/L in Milwaukee and Syracuse (Novotny *et al.* 1999). An urban stream receiving snow melt in Milwaukee had an average cyanide concentration of 31 µg/L (<2 – 45 µg/L). Two urban streams in Syracuse had average cyanide concentrations of 8 µg/L (<2 – 27 µg/L) and 48 µg/L (<2 – 167 µg/L), respectively. Reconsidering the NURP findings, three of the four cities which detected cyanide are within the snowbelt, and may have used deicing salts containing anti-caking agents. One (Austin, Texas) presumably does not.

In contrast to these relatively high concentrations associated with deicing salts, runoff from cities which do not use deicing salts or from northern cities outside the snow season has lower concentrations of cyanides. The City of Fresno NURP study (Brown & Caldwell, 1984) found undetectable cyanide (< 10 µg/L) in 19 grab samples of stormwater runoff from four watersheds with different land uses. Highway runoff from three urban sites in Michigan had average cyanide concentrations ranging from 5.8 – 9.3 µg/L. Samples were collected from June through October, which was outside the season where deicing salts might be used. Traffic volumes were high and ranged from 40,000 to 120,000 vehicles per day.

It is highly probable that the reported concentrations which exceed the freshwater acute guideline in urban stormwater are associated with the use of deicing salts containing the de-caking agent ferrocyanide. In situations where deicing salts are not being used, and where vehicle exhaust may be the dominant source, concentrations are much less (e.g., typically < 10 µg/L), even with high traffic volumes. Anti-caking agents will not be a source of cyanide in urban stormwater in the Project, and the forgoing discussion suggests that concentrations in stormwater runoff from the Project may reach concentrations of magnitude of approximately 10 µg/L, but are highly unlikely to exceed the acute CTR criteria of 22 µg/L.

The detectable concentrations observed in the Santa Clarita River at the mass emission station S29 (average of 10 µg/L) may be in part due to untreated urban stormwater runoff from the City of Santa Clarita. However, other sources are likely to be more significant. A potential source is cyanide from burnt catchments. For example, cyanide concentrations in run-off obtained from an area that had been burned in a wildfire that occurred in Tennessee and North Carolina averaged 49 µg/L (Barber *et al.* 2003). Higher cyanide concentrations were reported in runoff from a wild fire that occurred in New Mexico, with an average value of 80 µg/L.

In addition to the expected relatively low level of cyanide in untreated stormwater, cyanide in runoff from the Project would be readily removed by biological uptake, degradation by microorganisms, and by volatilization in the LID BMP and treatment control PDFs. Therefore cyanide is not expected to significantly impact the receiving waters of the Project.

7.3 New Development Regulatory Compliance

7.3.1 MS4 Permit Requirements for New Development as Defined in the SUSMP

PDFs include site design, source control, LID, and treatment control BMPs in compliance with the SUSMP requirements, as described in Section 5.1 and summarized in Table 5-1. LID BMP and treatment control PDFs will treat runoff from the entire urban portion of

the Project. Sizing criteria contained in the MS4 Permit and the SUSMP requirements will be met for all treatment control BMPs.

In summary, the proposed site design, source control, LID, and treatment control PDFs have been selected based on:

- effectiveness for addressing pollutants of concern in Project runoff, resulting in insignificant water quality impacts;
- sizing and outlet design consistent with the MS4 Permit and SUSMP requirements;
- additional design guidance consistent with the County of Los Angeles Department of Public Works Stormwater Best Management Practice Design and Maintenance Manual, other literature, and best professional judgment;
- hydrologic and water quality modeling to verify performance;
- meeting mean annual percent capture criteria contained in the California BMP New Development Manual; and
- providing specific O&M requirements to inspect and maintain the facilities.

On this basis, the proposed PDFs meet the MS4 Permit requirements for new development.

7.3.2 LID Performance Standard Retention Volume Conformance Analysis

The results of the LID Performance Standard retention volume conformance analysis demonstrate that the project exceeds the Project LID Performance Standard retention volume requirements (Table 7-18).

Table 7-18: LID Performance Standard Conformance Calculations

Step 1: Calculate LID Performance Standard [1]	
Project Area, ac	293
Project Impervious Area, ac	169
Allowable Effective Impervious Area,% of project	5%
Allowable Effective Impervious Area, ac	15
Remaining Impervious Area, ac	155
Runoff from Remaining Impervious Acre for 0.75 inch Storm, ac-ft	9.2

Step 2: Calculate Retention Volume in Parcel-based BMPs					
Land Use Treatment Categories	**Area, acres**	**Impervious Area, acres**	**BMP Area, acres**	**BMP Ret Depth, inches**	**BMP Retention Volume, ac-ft**
Category 1	40.0	31.1	1.2	18.3	1.9
Category 2	87.6	67.4	2.7	9.0	2.0
Category 3	0.0	0.0	0.0	1.8	0.0
Single Family HSCs	16.8	8.4	8.4	0.75	0.5
Parcel-based Total, ac-ft					4.4

Step 3: Calculate Retention Volume in Sub-Regional Infiltration/Biofiltration Facilities	
Sub-Regional Infiltration/ Biofiltration Facilities	**BMP Retention Volume[2], ac-ft**
Sub-regional Bioinfiltration Facilities	4.9
Sub-regional Biofiltration Facility	0.1
Sub-Regional Facility Total, ac-ft	5.0

Step 4: Compare Total Retention Volume to LID Performance Standard Retention Volume	
Total Volume Reduction achieved, ac-ft	9.4
LID Performance Standard Retention Volume, ac-ft	9.2
Surplus Retention Volume Provided, ac-ft	0.2

[2] Volume below lowest surface discharge, ac-ft

7.3.3 Los Angeles County LID Manual Requirements

Volume reductions provided by the Project's LID BMPs exceed the County LID Manual volume reduction requirements. A simple comparison of the volumetric retention requirements associated with the Project's LID Performance Standard and the County LID Manual's standard shows that the Project would achieve volume reductions exceeding those required by the County LID Manual. The project's LID BMPs are designed to fully retain the volume of stormwater runoff produced from a 0.75 inch storm event. The LID Manual requires that the excess volume (defined as the post-developed runoff volume minus the pre-developed runoff volume for the 0.75 inch storm event) be retained.

The Project LID Performance Standard allows impervious area up to 5 percent of the project area to be treated in media filters or equivalent where it is not feasible or practicable to implement infiltration, bioinfiltration, or biofiltration BMPs. The County LID Manual requires BMPs to be selected to infiltrate where feasible, but recognizes feasibility constraints that would limit infiltration. The result is that conformance with the Project LID Performance standard results in selection of BMPs that meet the BMP selection criteria of the County LID Manual and results in BMP sizes that are slightly larger than would be required to meet the sizing criteria of the County LID Manual.

7.4 Construction-Related Impacts

The potential impacts of construction activities, construction materials, and non-stormwater runoff on water quality during the construction phase are primarily due to sediment (TSS and turbidity) and certain non-sediment related pollutants. Construction-related activities that are primarily responsible for sediment releases are related to exposing previously stabilized soils to potential mobilization by rainfall/runoff and wind. Such activities include removal of vegetation from the site, grading of the site, and trenching for infrastructure improvements. Environmental factors that affect erosion include topographic, soil, and rainfall characteristics. Non sediment-related pollutants that are also of concern during construction relate to construction materials and non-stormwater flows and include construction materials (e.g., paint, stucco, etc); chemicals, liquid products, and petroleum products used in building construction or the maintenance of heavy equipment; and concrete-related pollutants.

Construction impacts due to Project development will be minimized through compliance with the Construction General Permit. This permit requires the discharger to perform a risk assessment for the proposed development (with differing requirements based upon the determined level) and to prepare and implement a Storm Water Pollution Prevention Plan (SWPPP), which must include erosion and sediment control BMPs that will meet or exceed measures required by the determined risk level of the Construction General Permit, as well as BMPs that control the other potential construction-related pollutants. A Construction Site Monitoring Program that identifies monitoring and sampling requirements during construction is a required component of the SWPPP. Preliminary analysis indicates that the Project will most likely be categorized as a Risk Level 2. BMPs required by the Construction General Permit will be incorporated assuming this level of risk; if final design analysis indicates that the Project will fall under Risk Level 3, the additional Level 3 permit requirements will be implemented as necessary.

7.4.1 Erosion and Sediment Control BMPs to be Implemented during Construction

Erosion control BMPs are designed to prevent erosion, whereas sediment controls are designed to trap or filter sediment once it has been mobilized. A SWPPP will be developed as required by, and in compliance with, the Construction General Permit and the City of Santa Clarita Standard Conditions. The General Permit requires the SWPPP to include BMPs to be selected and implemented based on the determine project risk level to effectively control erosion and sediment to the BAT/BCT. The following types of BMPs will be implemented as needed during construction:

Erosion Control

- Physical stabilization through hydraulic mulch, soil binders, straw mulch, bonded and stabilized fiber matrices, compost blankets, and erosion control blankets (i.e., rolled erosion control products).
- Limiting the area and duration (<14 days) of exposure of disturbed soils.
- Soil roughening of graded areas (through track walking, scarifying, sheepsfoot rolling, or imprinting) to slow runoff, enhance infiltration, and reduce erosion.
- Vegetative stabilization through temporary seeding and mulching to establish interim vegetation.
- Wind erosion (dust) control through the application of water or other dust palliatives as necessary to prevent and alleviate dust nuisance.

Sediment Control

- Perimeter protection to prevent sediment discharges (silt fences, fiber rolls, gravel bag berms, sand bag barriers, and compost socks).
- Storm drain inlet protection.
- Sediment capture and drainage control through sediment traps and sediment basins.
- Velocity reduction through check dams, sediment basins, and outlet protection/velocity dissipation devices.
- Reduction in off-site sediment tracking through stabilized construction entrance/exit, construction road stabilization, and entrance /exit tire wash.
- Slope interruption at permit-prescribed intervals (fiber rolls, gravel bag berms, sand bag berms, compost socks, biofilter bags).

Waste and Materials Management

- Management of the following types of materials, products, and wastes: solid, liquid, sanitary, concrete, hazardous and equipment-related wastes. Management measures include covered storage and secondary containment for material storage areas, secondary containment for portable toilets, covered dumpsters, dedicated and lined concrete washout/waste areas, proper application of chemicals, and proper disposal of all manners of wastes.
- Protection of soil, landscaping and construction material stockpiles through covers, the application of water or soil binders, and perimeter control measures.

- A spill response and prevention program will be incorporated as part of the SWPPP and spill response materials will be available and conspicuously located at all times on-site.

Non-Stormwater Management

- BMPs or good housekeeping practices to reduce or limit pollutants at their source before they are exposed to stormwater, including such measures as: water conservation practices, vehicle and equipment cleaning and fueling practices, and street sweeping. All such measures will be recorded and maintained as part of the project SWPPP.
- If construction dewatering or discharges from other specific construction activities such as water line testing, and sprinkler system testing are required, comply with the requirements of the Los Angeles RWQCB's General Waste Discharge Requirements (WDRs) under Order No. R4-2008-0032 (NPDES No. CAG994004) governing construction-related dewatering discharges.

Training and Education

- Inclusion of General Permit defined "Qualified SWPPP Developers" (QSD) and "Qualified SWPPP Practitioners" (QSP). QSDs and QSPs shall have required certifications and shall attend State Board sponsored training.
- Training of individuals responsible for SWPPP implementation and permit compliance, including contractors and subcontractors.
- Signage (bilingual, if appropriate) to address SWPPP-related issues (such as site cleanup policies, BMP protection, washout locations, etc).

Inspections, Maintenance, Monitoring, and Sampling

- Performing routine site inspections and inspections before, during (for storm events > 0.5 inches), and after storm events.
- Preparing and implementing Rain Event Action Plans (REAPs) prior to any storm event with 50% probability of producing 0.5 inches of rainfall, including performing required preparatory procedures and site inspections.
- Implementing maintenance and repairs of BMPs as indicated by routine, storm-event, and REAP inspections.
- Implementation of the Construction Site Monitoring Plan for non-visible pollutants, if a leak or spill is detected.
- Sampling of discharge points for turbidity and pH, at minimum, three times per qualifying storm event and recording and retention of results.

7.4.2 Construction BMP Implementation

During Project construction, BMPs will be implemented in compliance with the Construction General Permit and the general waste discharge requirements in the Dewatering General WDRs. The Project will reduce or prevent erosion and sediment transport and transport of other potential pollutants from the project site during the construction phase through implementation of BMPs meeting BAT/BCT in order to prevent or minimize environmental impacts and to ensure that discharges during the project construction phase will not cause or contribute to any exceedance of water quality standards in the receiving waters. All discharges from qualifying storm events will be sampled for turbidity and pH and results will be compared to Numeric Action Levels (250 NTU and 6.5-8.5, respectively) to ensure that BMPs are functioning as intended. If discharge sample results fall outside of these action levels, a review of causative agents and the existing site BMPs will be undertaken, and maintenance and repair on existing BMPs will be performed and/or additional BMPs will be provided to ensure that future discharges meet these criteria.

The construction-phase BMPs will assure effective control of not only sediment discharge, but also of pollutants associated with sediments, such as nutrients, heavy metals, and certain pesticides, including legacy pesticides. In addition, compliance with BAT/BCT requires that BMPs used to control construction water quality are updated over time as new water quality control technologies are developed and become available for use. Therefore, compliance with the BAT/BCT performance standard ensures mitigation of construction water quality impacts over time.

7.4.3 Compliance with Construction Permit and Construction Impacts

Prior to the issuance of preliminary or precise grading permits, the landowner or subsequent project applicant will provide the City Engineer with evidence that a Notice of Intent (NOI) has been filed with the State Water Resources Control Board. Such evidence will consist of a copy of the NOI stamped by the State Water Resources Control Board or Regional Water Quality Control Board, or a letter from either agency stating that the NOI has been filed and a copy of the site's applicable Waste Discharge identification (WDID) number.

Construction on the Project site may require dewatering. For example, dewatering may be needed if water has been standing on site and needs to be removed for construction, vector control, or other reasons. Further, dewatering may be necessary if groundwater is encountered during grading, or to allow discharges associated with testing of water lines, sprinkler systems and other facilities. In general, the Construction General Permit authorizes construction dewatering activities and other construction-related non-stormwater discharges as long as they (a) comply with Section III.C of the General Permit; (b) do not cause or contribute to violation of any water quality standards, (c) do

not violate any other provisions of the General Permit, (d) do not require a non-stormwater permit as issued by some RWQCBs, and (e) are not prohibited by a Basin Plan provision.

An additional Project Design Feature will be implemented to protect receiving waters from dewatering and construction related non-stormwater discharges. Such discharges will be implemented in compliance with the Los Angeles RWQCB's General Waste Discharge Requirements (WDRs) under Order No. R4-2008-0032 (NPDES No. CAG994004) governing construction-related dewatering discharges within the Project development areas. Typical BMPs for construction dewatering include infiltration of clean groundwater; on-site treatment using suitable treatment technologies; on-site or transport offsite for sanitary sewer discharge with local sewer district approval; or use of a sedimentation bag for small volumes of localized dewatering. Compliance with these WDRs constitutes a PDF, further assuring that the impacts of these discharges are not significant.

On this basis, the impact of Project construction-related runoff is considered less than significant.

7.5 Pollutant Bioaccumulation

Certain pollutants have the potential to accumulate in BMP vegetation and soils, potentially increasing the risk of exposure to wildlife and the food chain. Factors that could affect the extent of potential bioaccumulation include:

- The bioavailability of the pollutant;
- Conditions in the soils (*e.g.*, pH, acid-volatile sulfide concentration, organic content) that affect the form and bioavailability of the pollutant;
- The efficiency by which pollutants in the soils enter the plant community, the storage of these pollutants in plant tissues that are edible, and the utilization of the plants as a food source by animals;
- The type of habitats, organisms attracted to these habitats, and their feeding habits; and
- System design and maintenance.

The primary pollutants of concern with regard to bioaccumulation are mercury and selenium. However, as indicated by the water quality monitoring conducted by Los Angeles County at the Santa Clara River mass emission station S29 (LACDPW, 2005), selenium and mercury are not naturally present at levels of concern in this watershed.

Since these pollutants would not be introduced by the Project, bioaccumulation of selenium and mercury is not expected.

The potential for bioaccumulation impacts from the Project's LID and treatment control facilities, including bioretention, vegetated swales, and sub-regional infiltration/biofiltration basins, would be minimal. Since the tributary areas to the BMPs are largely impervious, very little coarse solids and associated pollutants are expected to be generated. The vegetation in the facilities would trap sediments and pollutants in the soils, which contain bacteria that metabolize and transform trace metals, thereby reducing the potential for these pollutants to enter the food chain. The facilities do not provide open water areas and are not likely to attract waterfowl.

Bioaccumulation of pollutants in the Santa Clara River would not be significant due to the low predicted concentrations of pollutants such as trace metals, which are predicted to be below the benchmark CTR criteria in the treated runoff. Also, sediments in the Santa Clara River are transported downstream in the wet season by storm flows, and therefore do not accumulate.
On this basis, the potential for bioaccumulation and adverse effects on waterfowl and other species is considered less than significant.

7.6 Dry Weather Runoff

Pollutants in dry weather flows could also be of concern because dry weather flow conditions occur throughout a large majority of the year, and because some of the TMDLs in downstream reaches of the Santa Clara River are applicable for dry weather conditions (e.g., nutrients and chloride).

Dry weather flows are typically low in sediment because the flows are relatively low and coarse suspended sediment tends to settle out or is filtered out by vegetation. As a consequence, pollutants that tend to be associated with suspended solids (e.g., phosphorus, some bacteria, some trace metals, and some pesticides) are typically found in very low concentrations in dry weather flows. The focus of the following discussion is therefore on constituents that tend to be dissolved, e.g., nitrate and trace metals, or constituents that are so small as to be effectively transported, e.g., pathogens and oil and grease.

In order to minimize the potential generation and transport of dissolved constituents, landscaping in public and common areas will utilize drought tolerant vegetation that requires little watering and chemical application. Landscape watering in common areas, commercial areas, multiple family residential areas, and in parks will use efficient irrigation technology utilizing evapotranspiration sensors to minimize excess watering.

In addition, educational programs and distribution of materials (source controls) will emphasize appropriate car washing locations (at commercial car washing facilities or the car wash pad in the multi-family residential areas) and techniques (minimizing usage of soap and water), encourage low impact landscaping and appropriate watering techniques, appropriate swimming pool dechlorination and discharge procedures, and discourage driveway and sidewalk washing. Illegal dumping will be discouraged by stenciling storm drain inlets and posting signs that illustrate the connection between the storm drain system and the receiving waters and natural systems downstream.

The LID BMPs will provide treatment for and infiltrate dry weather flows and small storm events. Water cleansing is a natural function of vegetation, offering a range of treatment mechanisms. Sedimentation of particulates is the major removal mechanism. However the performance is enhanced as plant materials allow pollutants to come in contact with vegetation and soils containing bacteria that metabolize and transform pollutants, especially nutrients and trace metals. Plants also take up nutrients in their root system. Pathogens would be removed through infiltration. Any oil and grease will be effectively adsorbed by the vegetation and soil within the LID BMPs. Dry weather flows and small storm flows will infiltrate in the LID BMPs.

The LID and treatment control PDFs will infiltrate or evapotranspire all expected dry weather runoff (see Section 7.9.2 below). It is expected that no dry weather discharge will occur to the Santa Clara River from the Project. Based on source control PDFs reducing the amount of dry weather runoff and treatment control PDFs capturing and treating the dry weather runoff that does occur, the impact from dry weather flows is considered less than significant.

7.7 Summary of Surface Water Quality Impacts

7.7.1 Direct Impacts

With the exception of runoff volume and chloride load (but not concentration), concentrations and loads of modeled constituents are predicted to decrease under proposed conditions when compared to existing conditions. Furthermore, modeled pollutant concentrations in runoff from developed areas with PDFs are below all benchmark water quality objectives and criteria and TMDL wasteload allocations for the Santa Clara River, are addressed by a comprehensive site design, source control, LID, and treatment control strategy, and compliance with MS4 Permit, Construction General Permit, and General De-Watering Permit requirements. In-stream concentrations of all modeled constituents are not predicted to increase under proposed conditions.

Concentrations of hydrocarbons are expected to increase, while concentrations of pathogens, pesticides, and trash and debris may or may not increase under proposed conditions when compared to existing conditions. None of the qualitatively assessed

constituents are expected to significantly impact receiving waters due to the implementation of a comprehensive site design, source control, LID, and treatment control strategy in compliance with the MS4 Permit, Construction General Permit, and General De-Watering Permit requirements. Therefore potential impacts from the Project on receiving water quality are not expected to be significant.

7.7.2 Cumulative Impacts

This section defines the geographic area of potential impact for the cumulative impacts analysis, and evaluates impacts from probable future projects together with the incremental effects of the proposed Project to determine effects on water quality and hydromodification within this geographic area. The model results presented below are used in addition to consideration of the other projects reflected in adopted plans and projections for areas tributary to Santa Clara River Reach 5 to get a better overall assessment of cumulative water quality effects on the Santa Clara River.

The geographic area for evaluating cumulative impacts includes the unincorporated area of Los Angeles County west of The Old Road to the Ventura County line. This geographic area includes the Newhall Ranch subregion, the Entrada project area, the Legacy Village project area, and the Valencia Commerce Center, as well as existing development in the Six Flags Magic Mountain area and the existing Valencia Water Reclamation Plant.

The proposed Entrada South Project (VTTM 53295) site is located west of Interstate 5 and the Old Road, south of Six Flags Magic Mountain Theme Park, north of the existing community of Westridge, and east of the boundary of the Newhall Ranch Specific Plan and the pending Mission Village, Vesting Tentative Tract Map 61105 (Figure 2-1). The Entrada South project proposes a total of 1,640 units including 408 single-family residences and 1,232 multi-family residences. The project will also include commercial areas, totaling 726,000 square feet of development, interchangeable for office or retail development, an elementary school, private drives, public facilities, a park, two private recreation centers, and natural and manufactured open space areas throughout.

The proposed Entrada North (VTTM 71377) project site is located northeast of the NRSP area and immediately west of Interstate 5 (Figure 2-1). A portion of the Entrada North project is located at the southwest corner of the intersection of Interstate 5 (I-5) and State Route 126 (SR-126) (at Castaic Junction) and the remaining portion lies north of Magic Mountain Parkway, west of I-5. The Entrada North project site lies east and north of the Six Flags Magic Mountain Theme Park. The Entrada North project proposes development of multi-family residential units and commercial/retail uses on approximately 454 gross acres.

The proposed Legacy Village project is located south of the NRSP area, bordering the Mission Village and Homestead Village projects within the NRSP, and north of the community of Stevenson Ranch. The 1,763 acre Legacy Village project proposes construction of residential areas and commercial space. Over 1,050 acres of open space will be incorporated into the Legacy Village project, including 25 acres of parks, recreational sites, and trails.

The remaining unbuilt portions of the Valencia Commerce Center in VTTM 18108 are located approximately one-half mile upstream of the confluence of Castaic Creek and the Santa Clara River. Approximately 4 million square feet of building floor area will be developed over the next five to ten years. Additionally, bank stabilization improvements to Castaic Creek and Hasley Creek would be constructed in conjunction with these remaining phases of the Commerce Center.

The NRSP LID Performance Standard described in Section 5 above would be implemented by the Entrada, Legacy Village, and Valencia Commerce Center projects, as well as the full suite of site design and source control BMPs, to address pollutants of concern in stormwater runoff and dry weather discharges from the proposed projects. Urban runoff from the Magic Mountain Theme Park and the Valencia WRP currently drains to the Santa Clara River and will continue to do so in proposed conditions without any anticipated change to stormwater management controls.

The combined effect on modeled pollutant loads and concentrations of the NRSP, Entrada South, Entrada North, Legacy Village, and the Valencia Commerce Center proposed projects are summarized in Tables 7-19 and 7-20 below, respectively. As shown in Table 7-19, when considered cumulatively, runoff volumes and loads of ammonia, dissolved copper, dissolved aluminum, and chloride are predicted to increase from the NRSP, Entrada, Legacy Village, and Valencia Commerce Center projects, while pollutant loads are expected to decrease for TSS, total phosphorus, nitrate-N + nitrite-N, total nitrogen, total lead, dissolved zinc, and total aluminum. Pollutant concentrations from the combined projects are predicted to decrease for all modeled parameters (Table 7-20). Increases in pollutant loadings are not anticipated to be significant based on the fact that predicted pollutant concentrations are well below benchmark water quality standards and TMDL wasteload allocations and are primarily within the range of observed concentrations in Santa Clara River Reach 5 (Table 7-21).

Table 7-19: Predicted Average Annual Combined Runoff Volume and Pollutant Loads for the NRSP, Legacy Village, Entrada, and Valencia Commerce Center Projects

Modeled Parameter	Units	Development Condition		Change
		Existing	Developed w/ PDFs	
Volume	acre-ft	1,245	3,968	2,723
Total Suspended Solids	tons	483	302	-181
Nitrate-N + Nitrite-N	tons	5.4	3.3	-2.1
Total Kjeldahl Nitrogen	tons	5.2	9.6	4.4
Total Nitrogen	tons	10.6	12.9	2.3
Total Phosphorus	tons	1.3	1.5	0.2
Total Aluminum	lbs	4,030	7,396	3,366
Dissolved Aluminum	lbs	732	1,508	776
Dissolved Copper	lbs	39	99	60
Total Lead	lbs	37	77	40
Dissolved Zinc	lbs	477	670	193
Chloride	tons	44	93	49

7-20: Predicted Average Annual Combined Pollutant Concentrations for the NRSP, Legacy Village, Entrada, and Valencia Commerce Center Projects

Modeled Parameter	Units	Development Condition		Change
		Existing	Developed w/ PDFs	
Total Suspended Solids	mg/L	285	56	-229
Nitrate-N + Nitrite-N	mg/L	3.2	0.6	-2.6
Total Kjeldahl Nitrogen	mg/L	3.1	1.8	-1.3
Total Nitrogen	mg/L	6.3	2.4	-3.9
Total Phosphorus	mg/L	0.8	0.3	-0.5
Total Aluminum	ug/L	1,191	685	-506
Dissolved Aluminum	ug/L	216	140	-76
Dissolved Copper	ug/L	12	9	-3
Total Lead	ug/L	11	7	-4
Dissolved Zinc	ug/L	141	62	-79
Chloride	mg/L	26	17	-9

Table 7-21: **Comparison of Predicted Pollutant Concentrations for the NRSP, Entrada, Legacy Village, and Commerce Center 26363 Projects with Water Quality Criteria and Observed Concentrations in Santa Clara River Reach 5**

Modeled Parameter	Units	Predicted Average Annual Concentration	TMDL/ Basin Plan Water Quality Objectives	California Toxics Rule Criteria[1]	Wasteload Allocations for MS4 Discharges into the Santa Clara River Reach 5	Range of Observed[2] Concentrations in Santa Clara River Reach 5
Total Suspended Solids	mg/L	56	Water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses	NA	NA	32 – 6,591
Nitrate-N + Nitrite-N	mg/L	0.6	5	NA	6.8[3]	0.5 – 4.8
Total Ammonia	mg/L	0.5	2.2[4]	NA	1.75[5]	<0.005 – 1.1
Total Nitrogen	mg/L	2.4	Waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance or adversely affects beneficial uses	NA	NA	<0.04 – 46[6]
Total Phosphorus	mg/L	0.3		NA	NA	0.18 – 13.4
Dissolved Copper	µg/L	9	NA	32	NA	3.3 – 22.6
Total Lead	µg/L	7	NA	260	NA	0.6 – 40
Dissolved Zinc	µg/L	62	NA	250	NA	3 – 37
Total Aluminum	µg/L	685	NA	750	NA	131 – 19,650
Chloride	mg/L	17	100	NA	100	3 - 121

[1] Hardness = 250 mg/L, based on minimum observed value at USGS Station 11108500. Lead criteria is for total recoverable lead. NAWQC aluminum criteria for pH 6.5 – 9.0.

[2] Range of concentrations observed in the Santa Clara River during wet weather (Stations S29, NR1, and NR3, see Section 2.3.1).

[3] 30-day average.

[4] 4-day average, ELS present, 90th percentile pH and temperature pairing observed at USGS Monitoring Station 11108500.

[5] 30-day average in Reach 5 below Valencia.

[6] Observed values for TKN (ammonia plus organic nitrogen).

NA – not applicable

As discussed above, the anticipated quality of effluent expected from the Landmark Village PDFs will not contribute concentrations of pollutants of concern that would be expected to cause or contribute to a violation of the water quality standards in the Project's receiving waters. Therefore, the Project's incremental effects on surface water quality are not expected to be significant.

The Landmark Village Project's surface runoff water quality, after PDFs, both during construction and post-development, is predicted to comply with adopted regulatory requirements that are designed by the LARWQCB to assure that regional development does not adversely affect water quality, including MS4 Permit and SUSMP requirements; Construction General Permit and General Dewatering Permit requirements; and benchmark Basin Plan water quality objectives, CTR criteria, and TMDLs. Any future urban development occurring in the Santa Clara River watershed must also comply with these requirements. By extrapolating the results of the direct and cumulative impact analysis modeling done for this Water Quality Technical Report, it can be predicted that analysis of other proposed development combined with existing conditions would have similar water quality results. Therefore, cumulative impacts on surface water quality of receiving waters from the Project and future urban development in the Santa Clara Watershed are addressed through compliance with the MS4 Permit and SUSMP requirements; Construction General Permit and General Dewatering Permit requirements; and benchmark Basin Plan water quality objectives, CTR criteria, and TMDLs, which are intended to be protective of beneficial uses of the receiving waters. Based on compliance with these requirements designed to protect beneficial uses, cumulative water quality impacts are mitigated to a level that is less than significant.

Impacts of Newhall Ranch Wastewater on Santa Clara River Water Quality and Hydrology

Wastewater generated by the Specific Plan build-out will be treated in the Newhall Ranch Water Reclamation Plant (WRP). The water quality and hydrologic impacts associated with the discharge of tertiary treated recycled water to the Santa Clara River were previously analyzed at the project-level in the Newhall Ranch Specific Plan Revised Draft EIR (Impact Sciences, 1999) as well as the Newhall Ranch Specific Plan Revised Additional Analysis (Impact Sciences, 2003).

The Newhall Ranch WRP treatment facility is further described in the individual NPDES Permit and Waste Discharge Requirements (WDRs) for the Newhall Ranch WRP (Order No. R4-2007-0046, effective October 27, 2007). Treatment at the WRP will consist of screening, activated sludge secondary treatment with membrane bioreactors, nitrification/denitrification, ultraviolet disinfection, and partial reverse osmosis. The initial design capacity of the WRP would be 2 million gallons per day (MGD) to accommodate the initial phases of the Newhall Ranch Specific Plan and would be

incrementally increased to 6.8 MGD to accommodate the sewage generated by the build-out of Specific Plan.

Treated effluent from the Newhall Ranch WRP would be used to supply distribution of recycled water throughout the Specific Plan area in the form of irrigation of landscaping and other approved uses. In an average rainfall year, all tertiary treated wastewater from the Newhall Ranch WRP would be recycled for irrigation and other non-potable uses, except in the wet weather months. During these months in average rainfall years, approximately 286 to 1,025 acre-feet of tertiary-treated wastewater would not be needed to meet estimated non-potable demand and, therefore, would be discharged to the Santa Clara River. During years 1 and 2 of the Newhall Ranch WRP operation, the WRP would operate at a maximum of 2 MGD, with an estimated average discharge flow rate of 0.2 MGD during the five month period of November through March. No sooner than year 3, the WRP would be expanded to 6.8 MGD, with an approximate average discharge flowrate of 0.6 MGD during this five month wet period. Therefore, discharge periods would coincide with peak wet months when dilution capacity is maximal (i.e., instream flows are highest). The average November through March instream flowrate at USGS station 11109000 (Newhall Bridge, approximately 2.5 miles downstream of the County line), is 188 cfs (121 mgd) based on measured average daily flow data for water years 1977-2006. Newhall WRP effluent would represent less than one percent of this average volume.

Order No. R4-2007-0046, NPDES Permit No. CA0064556 serves as the NPDES Permit for point source discharges from the Newhall Ranch WRP, pursuant to Section 402 of the federal Clean Water Act and Chapter 5.5, Division 7 of the California Water Code. The Order also serves as the Waste Discharge Requirements for the new Newhall Sanitation District with respect to discharges to the Santa Clara River, pursuant to Article 4, Chapter 4, of the California Water Code. Specifically, the Order specifies limitations and discharge requirements for the Newhall Ranch WRP, including discharge prohibitions, technology-based and water quality-based effluent limitations, receiving water limitations, and other provisions such as monitoring and reporting requirements.

The NPDES Permit contains effluent limitations that control the amount of conventional, non-conventional, and toxic pollutants discharged to the receiving waters. These effluent limits are a combination of technology-based limits (per 40 C.F.R. section 122.44(a)) and water quality-based limits (per 40 C.F.R. section 122.44(d)). Concentration-based effluent limitations contained in the NPDES Permit are listed in Table 7-22 below. Mass-based effluent limitations contained in the draft permit, but not listed in Table 7-22, are derived by multiplying the proposed concentration limitation by the permitted flow of 2.0 mgd. These mass-based limits would be modified accordingly as the phased plant upgrades approach completion following an anti-degradation analysis demonstration

conducted by Newhall Land, and upon certification and approval of increased treatment plant capacity.

Additional water quality-based effluent limits are included in the permit for toxicity in the WRP effluent and for temperature, pH, dissolved oxygen, fecal coliform, turbidity, toxicity, and other pollutants in the receiving water. Groundwater-based effluent limitations are included for coliform bacteria, chemical constituents, radionuclides, nitrate-N + nitrite-N, and taste or odor producing substances.

Title 22 of the California Code of Regulations (title 22) specifies California's Wastewater Reclamation Criteria (WRC) and all recycled water in California must meet or exceed these criteria to assure protection of receiving water quality. These criteria apply to the treatment processes; treatment performance standards, such as removal efficiencies and effluent water quality; process monitoring programs, including type and frequency of monitoring; facility operation plans; and necessary reliability features. The Newhall Ranch WRP discharges would be required to comply with the WRC through the issuance of a separate order.

As is discussed in the draft Newhall Ranch WRP NPDES Permit Fact Sheet (page F-14), the Upper Santa Clara River chloride wasteload allocations (WLAs) are expressed on a concentration basis derived from and equivalent to the existing water quality objective for Reaches 5 and 6 of the Santa Clara River, thereby providing direct protection for agricultural supply, the most sensitive beneficial use. Under the TMDL Implementation Plan, a special study was conducted to confirm that the concentration-based WLA of 100 mg/L chloride is protective of this beneficial use. A concentration-based WLA also accommodates future growth and provides beneficial uses protection from chloride loads that were in place at the time of the TMDL development. Protection of beneficial uses from additional chloride loads that were not assigned wasteload allocations is provided by using the WLAs as effluent limits in permits for new and future sources, such as the Newhall Ranch WRP.

7-22: Effluent Limitations in the Newhall Ranch WRP NPDES Permit

Parameter	Units	Effluent Limitations		
		Average Monthly	Average Weekly	Maximum Daily
Biochemical Oxygen Demand 5-day @ 20°C	mg/L	20	30	45
Total Suspended Solids	mg/L	15	40	45
pH	standard unit	6.5 - 8.5 (instantaneous minimum and maximum)		
Settleable solids	mL/L	0.1	-	0.3
Oil and Grease	mg/L	10	-	15
Total dissolved solids	mg/L	1,000	-	-
Chloride	mg/L	100 [1]	-	-
Sulfate	mg/L	400	-	-
Boron	mg/L	1.5	-	-
Total ammonia (NH_3 as N)	mg/L	1.93 [2]	-	3.87 [3]
Nitrate-N + Nitrite-N	mg/L	5	-	-
Nitrite-N	mg/L	0.9	-	-
Detergents (as MBAS)	mg/L	0.5	-	-
Total residual chlorine	mg/L	-	-	0.1
Antimony	µg/L	6	-	-
Arsenic	µg/L	10	-	-
Copper	µg/L	22	-	44
Lead	µg/L	13	-	26
Mercury	µg/L	0.051	-	0.10
Nickel	µg/L	100	-	-
Selenium	µg/L	4.1	-	8.2
Zinc	µg/L	5,000	-	-
Cyanide	µg/L	4.2	-	8.5
Acrylonitrile	µg/L	0.66		1.3
Tetrachloroethylene	µg/L	5	-	-
Bis(2-ethylhexyl)phthalate	µg/L	4	-	-
p-Dischlorobenzene (1,4-Dichlorobenzene)	µg/L	5	-	-
Lindane	µg/L	0.2	-	-
4,4-DDE	µg/L	0.00059	-	-
Iron	µg/L	300	-	-

[1] This is the water quality objective for chloride in the current Basin Plan. This effluent limitation is consistent with the assumptions of the Chloride TMDL for the Santa Clara River, Resolution No. 2002-018, Amendment to the Water Quality Control Plan for the Los Angeles Region to Include a TMDL for Chloride in the Santa Clara River (Chloride TMDL) and applies immediately. However, if a chloride site-specific objective (Chloride SSO) is adopted for the reach of the Santa Clara River in which Newhall Ranch WRP will discharge, then the permit may be reopened by the Los Angeles RWQCB to make the necessary changes, following USEPA approval of the Chloride SSO.

[2] This is the monthly average effluent limit calculated according to the Implementation Plan for ammonia in the Basin Plan, which specifies how to translate the Ammonia WQO into a final effluent limit, consistent with the assumptions of the Santa Clara River Nitrogen Compounds TMDL, Resolution No. 03-011.

[3] This is the daily maximum effluent limit calculated according to the Implementation Plan for ammonia in the Basin Plan, which specifies how to translate the Ammonia WQO into a final effluent limit, consistent with the assumptions of the Santa Clara River Nitrogen Compounds TMDL, Resolution No. 03-011.

Further stated in the Newhall Ranch WRP NPDES Permit Fact Sheet (page F-14), the Staff Report for the TMDL, dated August 21, 2002, states: "[a] concentration-based target accommodates future growth by allowing increased mass as long as it is accompanied by additional flow…. " The Fact Sheet finds that water quality would not be degraded if concentration-based wasteload allocations that are equivalent to the water quality objectives are assigned to new facilities at the end of pipe. The Fact Sheet also states that studies regarding the effect of additional chloride load on groundwater basins underlying the Upper Santa Clara River are underway and scheduled for completion by November 2007 (Fact Sheet page F-15). Initial results from these studies show that discharges at effluent limits of 100 mg/L chloride would not degrade groundwater quality. Results from these studies may be used to revise the effluent limits through modification of the NPDES permits for all dischargers discharging at 100 mg/L if necessary.

Similarly, concentration-based effluent limitations contained in the NPDES Permit for nitrogen compounds, established per the *Santa Clara River Nitrogen Compounds TMDL*, are protective of water quality in the Santa Clara River.

Water quality-based effluent limitations are included in the NPDES Permit for pathogen indicator bacteria as follows:

- E. Coli density shall not exceed 126/100 mL (geometric mean) or 235/100 mL (single sample);
- Fecal coliform density shall not exceed 200/100mL (geometric mean) or 400/100 mL (single sample).

These receiving water limitations are based on Resolution No. 01-018, Amendment to the Water Quality Control Plan for the Los Angeles Region to Update the Bacteria Objectives for Water Bodies Designated for Water Contact Recreation, and, therefore, are protective of beneficial uses in the Santa Clara River.

Based on required compliance with State and Federal water quality requirements, as discussed and analyzed in the project-level analysis contained in the Newhall Ranch Specific Plan Program EIR and the information above, and the implementation of previously adopted Mitigation Measures SP-5.0-52 through 5.0-56, which are related to the construction and operation of the WRP, no significant impacts related to discharge of Newhall Ranch recycled water would occur on Santa Clara River water quality or hydrology.

During an interim period until the first phase of the Newhall Ranch WRP is constructed, the Santa Clara Valley Sanitation District (SCVSD) will treat the wastewater from the

first 6,000 homes within the Specific Plan (up to 1.6 mgd) at the existing Valencia WRP as needed pursuant to the terms of a 2002 Interconnection Agreement. The wastewater from the first 6,000 homes within the Specific Plan is expected to produce wastewater chloride concentrations similar to those in the existing SCVSD service area; therefore, the interim discharge of wastewater from the Valencia WRP would not impact SCVSD's ability to comply with the adopted chloride TMDL. In addition, SCVSD has sufficient capacity to accommodate the interim use of its facilities. Newhall also has voluntarily agreed to incorporate interim chloride reduction treatment at the Valencia WRP to confirm compliance with the chloride TMDL. On that basis, no significant impacts related to the interim discharge of Newhall Ranch recycled water from the Valencia WRP would occur on the Santa Clara River water quality or hydrology.

7.8 Groundwater Impacts

7.8.1 Direct Groundwater Quality Impacts

Discharge from the Project's developed areas to groundwater will occur in three ways: (1) through general infiltration of irrigation water, (2) through incidental infiltration of urban runoff in the proposed treatment control PDFs after treatment, and (3) through infiltration of urban runoff, after treatment in the Project PDFs, in the Santa Clara River, which is the primary recharge zone for groundwater in the Santa Clara Valley. Groundwater quality will be fully protected through implementation of the Project's site design, source control, and treatment control PDFs prior to discharge of Project runoff to groundwater.

Per the LARWQCB Clarification Letter (LARWQCB, 2006), generally, the common pollutants in stormwater are filtered or adsorbed by soil, and unlike hydrophobic solvents and salts, do not cause groundwater contamination. In any case, infiltration of one to two inches of rainfall in semi-arid areas like Southern California where there is a high rate of evapotranspiration presents minimal risks.

The pollutant of concern with respect to groundwater is nitrate-N plus nitrite-N. The Basin Plan groundwater quality objective for nitrate-nitrogen plus nitrite-nitrogen is 10 mg/L (which is more stringent than the objective for nitrate-nitrogen alone (10 mg/L) and for nitrite-nitrogen alone (1 mg/L)). The predicted nitrate-nitrogen plus nitrite-nitrogen concentration in runoff after treatment in the PDFs is 0.6 mg/L, which is well below the groundwater quality objective.

Treated effluent from the Newhall Ranch WRP will be used to supply distribution of recycled water throughout the Specific Plan area in the form of irrigation of landscaping and other approved uses. The effluent limitation contained in the Newhall Ranch WRP Permit for nitrate-N plus nitrite-N is 5 mg/L and the limitation for nitrite-N is 0.9 mg/L (average monthly). As the Basin Plan groundwater quality objective for nitrate-nitrogen

plus nitrite-nitrogen is 10 mg/L or 1 mg/L for nitrite-nitrogen, the Newhall Ranch WRP irrigation water supply that will serve the Project will be well below the groundwater quality objectives.

On this basis, the potential for the Project to adversely affect groundwater quality is considered less than significant.

7.8.2 Cumulative Groundwater Quality Impacts

As discussed above, the anticipated quality of runoff discharges from the Project's developed areas and irrigation to groundwater will not contribute loads or concentrations of pollutants of concern that would be expected to cause or contribute to a violation of the groundwater quality standards. By extrapolating these results to existing and proposed development throughout the watershed and based on a review of adapted plans and projections, it is concluded that no adverse cumulative effects would occur to groundwaters. Therefore, the Project's incremental effects on groundwater quality are not expected to be significant.

The Project's discharges to groundwater, after PDFs, both during construction and post-development, is predicted to comply with adopted regulatory requirements that are designed by the LARWQCB to assure that regional development does not adversely affect water quality, including MS4 Permit and SUSMP requirements; Construction General Permit and General Dewatering Permit requirements; and benchmark Basin Plan groundwater quality objectives. Any future urban development occurring in the Santa Clara River watershed must also comply with these requirements. Therefore, cumulative impacts on groundwater quality from the proposed Project and future urban development in the Santa Clara Watershed are addressed through compliance with the MS4 Permit and SUSMP requirements, Construction General Permit requirements, General Dewatering Permit requirements, and benchmark Basin Plan groundwater quality objectives, which are intended to be protective of beneficial uses of the groundwater. Based on compliance with these requirements designed to protect beneficial uses, cumulative groundwater quality impacts are mitigated to a level that is less than significant.

7.8.3 Groundwater Recharge Impacts

Direct Project Impacts

In a groundwater basin, the effect of urbanization on recharge to underlying groundwater is dependent on land uses, water uses, vegetative cover, and geologic conditions. Groundwater recharge from undeveloped lands occurs from precipitation alone, whereas areas that are developed for agricultural or urban land uses receive both precipitation and irrigation of vegetative cover. In an urban area, groundwater recharge occurs directly beneath irrigated lands and in drainages whose bottoms are not paved or cemented. A

memorandum prepared by CH2M Hill entitled "Effect of Urbanization on Aquifer Recharge in the Santa Clarita Valley" (Appendix E) discusses the general effects of urbanization on groundwater recharge and the specific effects in the Santa Clarita Valley.

Currently the site is irrigated agricultural land. As a result, in the existing condition recharge occurs within the Project site from irrigation and precipitation. On one hand, development of the site will introduce impervious surface over approximately 61 percent of the tract map site, which will tend to reduce recharge. In addition, development of agricultural lands will eliminate agricultural irrigation as a source of recharge. On the other hand, development of the site will increase runoff volume discharged after treatment to the Santa Clara River, whose channel is predominantly natural and consists of vegetation and coarse-grained sediments (rather than concrete). The porous nature of the sands and gravels forming the streambed will allow for significant infiltration to occur to the underlying groundwater. Also, the Project will introduce landscaping, irrigation, and PDFs designed to infiltrate runoff. These project effects will increase groundwater recharge from the Project. On balance, it is unlikely that the Project will result in a significant change in groundwater recharge in the project vicinity. Based on the above discussion, the Project's impact on groundwater recharge is considered less than significant.

Cumulative Impacts

Increased urbanization in the Santa Clarita Valley has resulted in the irrigation of previously undeveloped lands. The effect of irrigation is to maintain higher soil moisture levels during the summer than would exist if no irrigation were occurring. Consequently, a greater percentage of the fall/winter precipitation recharges groundwater beneath irrigated land parcels than beneath undeveloped land parcels. In addition, urbanization in the Santa Clarita Valley has occurred in part because of the importation of State Water Project (SWP) water, which began in 1980. SWP water use has increased steadily, reaching nearly 44,500 acre-feet (AF) in 2003. Two-thirds of this water is used outdoors, and a portion of this water eventually infiltrates to groundwater. The other one-third is used indoors and is subsequently routed to local water reclamation plants (WRPs) and then to the Santa Clara River (after treatment). A portion of this water flows downstream out of the basin, and a portion infiltrates to groundwater.

Records show that groundwater levels and the amount of groundwater in storage were similar in both the late 1990s and the early 1980s, despite a significant increase in the urbanized area during these two decades. This long-term stability of groundwater levels is attributed in part to the significant volume of natural recharge that occurs in the streambeds, which do not contain paved, urban land areas. On a long term historical basis, groundwater pumping volumes have not increased due to urbanization, compared with pumping volumes during the 1950s and 1960s when water was used primarily for agriculture. Also, the importation of SWP water is another process that contributes to

recharge in the Valley. In summary, urbanization has been accompanied by long-term stability in pumping and groundwater levels, plus the addition of imported SWP water to the Valley, which together have not reduced recharge to groundwater, nor depleted the amount of groundwater that is in storage within the Valley.

Based on the above discussion, the cumulative impact on groundwater recharge is considered less than significant.

7.9 Hydromodification Impacts

Development typically increases impervious surfaces on formerly undeveloped (or less developed) landscapes, reducing the capture and infiltration of rainfall. The result is that, as a watershed develops, a larger percentage of rainfall becomes runoff during any given storm. In addition, runoff reaches the stream channel more efficiently due to the development of storm drain systems, so that, if no controls are implemented, the peak discharge rates for rainfall events and floods are higher for an equivalent event than they were prior to development. Further, the introduction of irrigation and other dry weather flows can change the seasonality of runoff reaching natural receiving waters. These changes, in turn, affect the stability and habitat of natural drainages, including the physical and biological character of these drainages. This process, termed "hydromodification" (SCCWRP, 2005a) is addressed in this section.

Significant adverse hydromodification impacts are presumed to occur if the proposed Project would:

- Substantially alter the existing drainage pattern of a natural drainage, stream, or river causing substantial erosion, siltation, or channel instability; or
- Substantially increase the rates, velocities, frequencies, duration and/or seasonality of flows causing channel instability and harming sensitive habitats or species in natural drainages in a manner that substantially adversely affects beneficial uses.

All flows from those areas of the Project that will be developed with impervious surface with potential for altering drainage patterns will be discharged directly to the Santa Clara River. Therefore, this analysis addresses the potential for hydromodification impacts to the Santa Clara River as a result of the proposed Project. The impervious surfaces associated with the proposed water tanks are very minor and will not alter drainage patterns, and therefore no potential for hydromodification impacts exists from these areas of the Project.

The physical alteration of natural drainages, such as bank protection, energy dissipaters, and bridge abutments, are not impacts created by changes in runoff seasonality, volume,

duration, or flow associated with development. Instead, these types of alterations are physical alterations to the stream bed and bank, with associated effects on stream habitat and species. These types of effects are analyzed in the Landmark Village Draft EIR and more specifically the biological chapter of the EIR for this Project.

7.9.1 Wet Weather Flows

Direct Impacts to the Santa Clara River

The Project proposes development of approximately 80 percent (233 acres) of the 292.6 acre tract map site; the remaining 59.6 acres would be used for trails, parks, and vegetated slopes and water quality BMPs. Overall, approximately 61 percent (178.4 acres) of the tract map area would be impervious and 39 percent (114.2 acres) would be vegetated. The size of the Project in comparison to both the 1,618 square mile total watershed area and the expected total impervious area in the watershed in the existing conditions and at build-out is small. It is estimated, based on the land use data provided by LACDPW, that the proposed Project will comprise 0.5 percent of the total impervious area in the watershed encompassing the Project location at ultimate planned build-out for the watershed. See Section 4.4.3 above for information regarding adopted plans and projection used to derive build-out assumptions for the watershed.

A series of progressive hydromodification control measures will be used in the Project to prevent and control hydromodification impacts to the Santa Clara River:

- Avoid, to the extent possible, the need to mitigate for hydromodification impacts by preserving natural hydrologic conditions and protecting sensitive hydrologic features, sediment sources, and sensitive habitats.

- Minimize the effects of development through site design practices (e.g., reducing connected impervious surfaces and providing river buffers) and implementation of stormwater volume-reducing BMPs (project-based hydrologic source control).

- Mitigate hydromodification impacts in-stream using a geomorphically-based approach (e.g., buried soil cement bank stabilization).

Project-based Hydrologic Source Control

Disconnecting impervious areas from the drainage network and adjacent impervious areas is a key approach to protecting channel stability. Several hydrologic source controls will be included in the Project that will limit impervious area and disconnect imperviousness:

Site design. Site design will help to reduce the increase in runoff volume, including the clustering of NRSP development into village areas, including the Landmark Village, the preservation of 70 percent of the NRSP area in open space, and 59.6 acres (20 percent) of the Project in trails, slopes, and LID BMPs; routing of impervious area runoff to vegetated areas; use of native and/or non-native, non-invasive mostly drought tolerate plants in landscaped areas; and the use of efficient irrigation systems in common area landscaped areas. These measures will help to protect the stability of the Santa Clara River and to avoid and minimize direct impacts to the River.

LID BMPs and Treatment Controls. The Project's LID BMPs and treatment control will also serve as hydromodification source control BMPs. Vegetated BMPS such as vegetated swales, filter strips, and bioretention areas with underdrains can provide volume reduction on the order of 20 to 30 percent through infiltration and evaporation. If site conditions are suitable, all water captured in the LID BMPs would be effectively removed from the project's stormwater discharges. Collectively, these LID BMPs are expected to provide significant reduction in wet weather runoff. In addition, these facilities will also receive and eliminate dry weather flows.

Geomorphically-Referenced Channel Design

The hydromodification management approach for the Santa Clara River will incorporate "geomorphically-referenced river engineering" as described in SCCWRP Technical Report 450 (SCCWRP, 2005a). The goal of this approach is to preserve the appearance of the natural stream channel to the maximum extent practicable while maintaining stability in stream channel morphology. The Project's development footprint will allow for the greatest freedom possible for "natural stream channel" activity. This includes establishing buffer zones and maintaining setbacks to allow for channel movement and adjustment to changes in energy associated with runoff. The engineered structural elements that will be implemented where needed for the Santa Clara River include energy dissipation and bank stabilization.

Energy Dissipation. Energy dissipation at storm drain outfalls provides erosion protection in areas where discharges have the potential to cause localized stream erosion. Erosion protection will be provided at all storm drain outlets to the Santa Clara River.

Bank Stabilization. The Project will include buried soil cement along the Santa Clara River and Castaic Creek adjacent to and downstream of the Project site where necessary to protect against flooding and erosion pursuant to Federal Emergency Management Administration (FEMA) and Los Angeles County Department of Public Works' requirements. In total, approximately 18,600 LF of bank would be provided with buried soil cement protection. This would include approximately 11,000 feet fronting the tract map site and approximately 6,400 LF on the south bank downstream (west) of the Long

Canyon Road Bridge. The alignment was selected so that bank protection along the river would generally be excavated from non-jurisdictional upland areas adjacent to the river. Installing bank protection in non-jurisdictional areas reduces and/or avoids impacts to the river, and has the potential to create new riverbed areas, allows for channel movement and adjustment to changes in energy associated with runoff, and increases riparian habitat.

Additional buried bank stabilization would be constructed as part of the approved Newhall Ranch Water Reclamation Plant (WRP) and between the Old Road and the Santa Clara River (protecting the utility corridor). The bank protection between the Old Road and the Santa Clara River was approved as part of the Santa Clara River Natural River Management Plan (NRMP).

Approximately 6,600 LF of Turf Reinforcement Mat (TRM) or similar bank stability protection would be provide along the southern edge of the utility corridor downstream or west of the tract map site. TRMs are designed to reinforce vegetation at the root and stem allowing vegetation to be used as erosion control in areas where flow conditions exceed the ability of natural vegetation to remain rooted. This includes applications with high slopes or stream banks where grouted rip-rap and concrete channels are aesthetically undesirable.

Conclusion

In summary, although Project runoff volumes, flow rates, and durations will increase, potential impacts of hydromodification (i.e., the potential to cause erosion, siltation, or channel instability) will be minimized by the Project PDFs. The Project's site design and LID BMP PDFs will minimize increases in runoff volume from the development area, the preferred method for controlling hydromodification impacts from new development (SCCWRP, 2005a).

Potential instream impacts of increased volumes, rates, and flow durations will be managed and mitigated with energy dissipaters at the discharge points to the Santa Clara River and the River banks will be protected with vegetated buried bank stabilization primarily in non-jurisdictional upland areas adjacent to the river. This type of technique is the preferred approach for bank stabilization (SCCWRP, 2005a).

For these reasons, the wet weather direct hydromodification impacts of the Project with PDFs on the Santa Clara River are considered less than significant.

Cumulative Impacts

As identified in the MS4 Permit, increased volume, velocity, and discharge duration of stormwater runoff from the cumulative existing and future developed areas in watersheds

of natural drainages, including the Santa Clara River, has the potential to accelerate downstream erosion and impair stream habitat. Given the very large size of the Santa Clara River watershed, the contribution of the Project to cumulative hydromodification impacts to the Santa Clara River is difficult to assess quantitatively. Therefore, a qualitative assessment that references total predicted development per adopted General Plans and projections for the Santa Clara River watershed is provided below.

Effect of Watershed Impervious Area

The limited hydromodification impact research to date has focused on empirical evidence of channel failures in relationship to directly connected impervious area (DCIA) or total impervious area. However, more recent research has established the importance of size of watershed, channel slope and materials, and climatic and precipitation patterns (SCCWRP 2005a, Balance Hydrologics 2005 (provided in Appendix F)). Impervious area that drains directly to a storm drain system and then to the receiving water is considered "directly connected," whereas impervious area that drains through vegetation or to infiltration facilities is considered "disconnected."

Booth and Jackson (1997) reported finding a correlation between loss of channel stability and increases in DCIA. In Washington State, streams were found to display the onset of degradation when the DCIA increases to ten percent or more, and a lower imperviousness of five percent was found to cause significant degradation in sensitive watersheds (Booth and Jackson, 1997). The Center for Watershed Protection (Schuler and Holland, 2000) described the impacts of urbanization on stream channels and established thresholds based on total imperviousness within the tributary drainage area. It states "a threshold for urban stream stability exists at about 10 percent imperviousness." It further states that a "sharp threshold in habitat quality exists at approximately 10 percent to 15 percent imperviousness." These studies, however, addressed changes in a very different climatic region than Southern California.

Geosyntec's work in the San Francisco Bay area's Santa Clara Valley (Geosyntec 2004) also evaluated the relationship between imperviousness and stream channel degradation in an area that had predominately directly connected impervious areas. Geosyntec found similar results to those published by Booth and Schuler, where channel erosion was observed at approximately six to nine percent imperviousness for two separate watershed systems. More recent studies conducted by Geosyntec in this same watershed area showed that levels as low as two to three percent total imperviousness could lead to stream channel degradation, depending on channel characteristics. This region also has different climatic characteristics than Southern California.

Although physical degradation of stream channels in semi-arid climates of California may be detectable when watershed imperviousness is between three and five percent, not

all streams will respond in the same manner (SCCWRP, 2005b). Management strategies need to account for differences in stream type, stage of channel adjustment, current and expected amount of basin imperviousness, and existing or planned hydromodification control strategies.

The absolute measure of watershed imperviousness that could cause stream instability in the Santa Clara River depends on many factors, including watershed area, land cover, and soil type; development impervious area and connectedness; reduced sediment yield; longitudinal slope of the river; channel geometry; and local boundary materials, such as bed and bank material properties and vegetation characteristics. Based on land use data provided by the County of Los Angeles (see Section 4.4.3 above), the estimated cumulative level of percent impervious area at build-out in the Santa Clara River watershed upstream from the NRSP area is nine percent.

Effect of Catchment Drainage Area

The Southern California Coastal Water Research Project (SCCWRP) found signs of hydromodification impacts in Southern California streams when watershed percent imperviousness was around two to three percent for streams with a catchment drainage area of less than five square miles (mi^2) (SCCWRP, 2005a). Recognizing that their findings were based on the type and size of catchments that were measured, the researchers in the SCCWRP study attempted to develop a framework by which their results could be extended to other stream types. They developed a classification system based on watershed characteristics, stream channel characteristics (including level of vegetative development), and stream channel resistance, and suggested these features could be important in selecting management strategies and approaches to control hydromodification impacts. The Level 1 classification is based on watershed characteristics that include the size, shape, and topography of the watershed.

The catchment drainage area (CDA) is stated to be the most obvious differentiator among watersheds, as this is likely to have the greatest effect on runoff. The SCCWRP study focused on small watersheds (< 5 mi^2), whereas the CDA of the Santa Clara River at the Los Angeles County line, near the western edge of the NRSP area (the Upper Watershed), is about 640 mi^2. Based on the differences in CDA, the SCCWRP findings with respect to CDA would not be applicable to the Santa Clara River. Information in the SCCWRP report, based in part on the work of Zielinski (2002), suggests that smaller watersheds are more responsive and sensitive to changes in land use, whereas larger watersheds (> 30 mi^2) were said to be less responsive to land use changes. Geosyntec's work in the San Francisco Bay area found significant hydromodification impacts on streams of watersheds that were 40 mi^2 in size; however, this is still substantially smaller than the Santa Clara River watershed at the Los Angeles County line. Given the large CDA for the Santa Clara River, the river is likely less responsive to potential

hydromodification effects, but channel morphology must still be examined to determine the level and potential significance of Santa Clara River response.

Application to the Santa Clara River

Balance Hydrologics assessed the potential effects of the planned cumulative urbanization within the Santa Clara River upstream of the County line (the upper watershed) on channel morphology by examining historical changes in the Santa Clara River channel pattern in response to different types of major disturbance using historical rainfall and other relevant records and aerial channel photography (Balance Hydrologics, 2005 (provided in Appendix F)). The findings of this analysis are summarized below.

The Santa Clara River is a dynamic, episodic system. Understanding the magnitude of geomorphic change over the course of recent history in response to natural and human disturbances in the watershed is a key factor in assessing the potential response to future urbanization within the watershed.

For example, the report examines the construction of Castaic Dam in the 1974 (affecting approximately 30 percent of the Santa Clara River watershed above Castaic Creek), which cut off a significant supply of sediment to the Santa Clara River. This change, however, does not appear to have had an effect on the channel dimensions of the Santa Clara River mainstem. The width of the active corridor as well as the general form of the channel are generally consistent before and after construction of the dam. It appears that the Santa Clara River had enough buffering capacity to absorb this change. The report finds that the depletion of sediment supply to the mainstem, which would typically be expected to cause erosive effects, did not, in fact, result in those effects, perhaps because reductions in sediment were offset by additional available sediment stored in the basin in the upper watershed as a result of movement along the San Gabriel fault.

Similarly, the report examines the amount of vegetation within the Santa Clara River corridor, which appears to have generally increased since the 1960s, likely due to the increase in available summer flows due to the Valencia and Saugus Water Reclamation Plants' discharges. However, this vegetation does not seem to provide enough erosion resistance to maintain a "stable" channel capable of withstanding regular 're-sets', large events that completely alter the form of the Santa Clara River channel which occur at intervals averaging about a decade, or much less than the expected lifetime of the riparian woodlands which do get established. Despite heavy vegetation on the channel banks near the NRSP area and in areas of ground-water upwelling, the stream still responds to large events by a general widening and/or shift of the active channel within the River corridor.

After studying the response of the river to several different anthropogenic and natural disturbances, the report concludes that the Santa Clara River, as with many streams in

semi-arid southern California, is highly episodic. Concepts of "normal" or "average" sediment-supply and flow conditions have limited value in this "flashy" environment, where episodic storm and wildfire events have enormous influence on sediment and storm flow conditions. In these streams, a large portion of the sediment movement events can occur in a matter of hours or days. Other perturbations which can potentially affect channel geometry appear to have transitory or minor manifestations. For example, effects on the channel width due to the 1980s levee construction were barely discernible by the first few years of the 21st century, probably mostly due to morphologic compensation associated with the storm events in the mid- to late-1990s. As a result, channel morphology, stability, and character of the Santa Clara River is almost entirely determined by the "reset" events that occur within the watershed.

Fluvial Study

Additional study of the Santa Clara River has been performed by Pacific Advanced Civil Engineering, Inc., who prepared a comprehensive fluvial analysis for Santa Clara River through the NRSP area (PACE, 2006) for LACDPW. A river fluvial analysis is the study of the river bed and bank sediment movement over time and as a result of flow in the river and changes in the tributary watershed.

The fluvial analysis had three distinct components:

1. Analysis of long term trends of river bed and bank sediment build-up (aggredation) or removal (degradation) was performed. More than 80 years of available historic topographic mapping of the river indicated no real trend of aggredation or degradation in the study reach.

2. General (capital storm event) aggredation/degradation calculations were performed to determine the expected fluvial response of the river to the LACDPW design storm event (>140,000 cfs). US Army Corps of Engineers computer modeling software (SAM) was used to evaluate existing and proposed project conditions. Only minor variations in the fluvial response were shown in the modeling.

3. Local aggredation/degradation resulting from river curvature, bridges, river bed material, and various other components were considered and estimates of aggredation and degradation were calculated.

To complete the fluvial analysis, long term, general, and local aggredation/degradation components were added together to obtain the total aggredation/degradation for each river section within the study reach.

One of the purposes for the fluvial analysis, which has been approved by LACDPW, was to provide a level of understanding of the Santa Clara River Newhall Ranch reach fluvial mechanics related to existing conditions and proposed NRSP development conditions to identify any potential project impacts. The fluvial analysis showed very little change between the pre- and post-development conditions and therefore concluded that there is no potential adverse impact to the fluvial mechanics of the river.

Conclusion

As discussed above, the Project will include a number of hydrologic source control PDFs that will substantially lessen any potential contribution to cumulative hydromodification impacts to the Santa Clara River. In addition, it is presumed that all future development within the NRSP, Legacy, and Entrada sub-regions will implement hydromodification controls consistent with the NRSP Sub-regional Stormwater Mitigation Plan. Further, other future projects within the watershed reflected in adopted plans and projections will implement hydromodification controls to meet flow criteria that will be adopted by the LACDPW under Part 4, § D.1 of the MS4 Permit. These measures are designed to mitigate and prevent direct and cumulative hydromodification impacts.

Within the Santa Clara River watershed, major perturbations (urbanization, dam construction, levee construction, decadal changes in climate, and increases in woody vegetation) do not appear to have had a significant impact on the geomorphic expression of the Santa Clara River. Large "re-set" events (those which are typically not as affected by increases in impervious area) have episodically completely altered the form of the Santa Clara River channel. These events, occurring on average once every ten years, are a dominant force in defining channel characteristics. The geomorphic dominance of "re-set" events determines the geomorphic character of the Santa Clara River and the Santa Clara River's response to anthropogenic perturbations, including hydromodification impacts associated with development, is expected to be minimal in light of the "re-set" driven nature of the Santa Clara River channel. Due to these episodic "re-sets," "unraveling" of the Santa Clara River mainstem due to hydromodification associated with cumulative urban development within the watershed, as is seen in many smaller southern California watersheds, is not expected to occur. The "re-set" events appear to adequately buffer changes that may occur in short-term sediment transport between re-set events.

Based upon the above discussion, that the Project includes hydromodification controls as Project Design Features, that future development projects within the watershed will control flow in compliance with the regional program, and that large-scale changes naturally occur in the Santa Clara River in response to major episodic events, the Project's contribution to cumulative hydromodification impacts to the Santa Clara River will be less than significant and consistent with the requirements of the MS4 permit.

7.9.2 Dry Weather Runoff

Direct Impacts

It is predicted that all dry weather flows will be infiltrated or removed by evapotranspiration in the LID BMP PDFs, which also provide hydrologic source control. As a result, no appreciable change in seasonality of flows is anticipated to result from development. Based on comprehensive site planning, source control, LID, and treatment control strategy and the water balance analysis, the impact of the projects within the Newhall Ranch subregion on dry weather water quality and seasonality of flow in the tributaries and the River is considered less than significant.

8 CONCLUSIONS

This section summarizes the potential effects, if any, of the proposed Landmark Village Project on water quality and hydromodification in Santa Clara River Reach 5.

8.1 Water Quality Impacts

The following are the conclusions regarding the significance of impacts for the pollutants of concern under wet and dry weather conditions:

- ***Sediments*:** MS4 Permit, Construction General Permit, Dewatering General Permit, and SUSMP-compliant BMPs will be incorporated into the Project to address sediment in both the construction phase and post-development. Mean total suspended solids concentration and load are predicted to be less in the post-development condition than in the existing conditions. Turbidity in stormwater runoff will be controlled through implementation of a Construction SWPPP and will be permanently reduced through the stabilization of erodible soils with development. On this basis, the impact of the Project on sediments is considered less than significant.
- ***Nutrients (Phosphorus and Nitrogen (Nitrate-N+Nitrite-N, Ammonia-N, and Total Nitrogen))***: MS4 Permit, Construction General Permit, Dewatering General Permit, and SUSMP-compliant BMPs will be incorporated into the Project to address nutrients in both the construction phase and post-development. Total phosphorus, nitrate-nitrogen plus nitrite-nitrogen, ammonia-nitrogen, and total nitrogen concentrations and loads are predicted to decrease in the post-developed condition and to be within the range of observed values in Santa Clara River Reach 5. Nitrate-N plus nitrite-N and ammonia-N concentrations are predicted to decrease with development to a point well below Basin Plan objectives and TMDL wasteload allocations. The predicted total nutrient concentrations are not expected to cause increased algal growth. On this basis, the impact of the Project on nutrients is considered less than significant.
- ***Trace Metals*:** MS4 Permit, Construction General Permit, General Dewatering Permit, and SUSMP-compliant BMPs will be incorporated into the Project to address trace metals in both the construction phase and post-development. The mean loads and concentrations of dissolved copper, total lead, and dissolved zinc, and total aluminum are predicted to decrease with Project development. Mean concentrations of dissolved copper, total lead, dissolved zinc, and total aluminum are predicted to be below benchmark Basin Plan objectives, CTR criteria, and the NAWQC criterion for aluminum. Cadmium is not expected to be present at significant levels in Project runoff. On this basis, the impact of the Project on trace metals is considered less than significant.

- ***Chloride*:** MS4 Permit, Construction General Permit, Dewatering General Permit, and SUSMP-compliant BMPs will be incorporated into the Project to address chloride loads (via volume reduction) in both the construction phase and post-development. The mean concentration of chloride is predicted to decrease with development, while the average annual load is predicted to increase. The predicted concentration is well below the Los Angeles Basin Plan objective and the TMDL wasteload allocation and is within the range of observed values in Santa Clara River Reach 5. On this basis, the impact of the Project on chloride is considered less than significant.
- ***Pesticides*:** Pesticides in runoff may or may not increase in the post-development phase as a result of landscape applications. Proposed pesticide management practices, including source control, removal with sediments in LID BMP and treatment control PDFs, and advanced irrigation controls, in compliance with the requirements of the MS4 Permit and the SUSMP Manual, will minimize the presence of pesticides in runoff. During the construction phase of the Project, erosion and sediment control BMPs implemented per General Construction Permit and General De-Watering Permit requirements will prevent pesticides associated with sediment from being discharged. Final site stabilization will limit mobility of legacy pesticides that may be present in pre-development conditions. On this basis, the impact of the Project on pesticides is considered less than significant.
- ***Pathogens*:** Post-development pathogen sources include both natural and anthropogenic sources. The natural sources include bird and mammal excrement. Anthropogenic sources include leaking septic and sewer systems and pet wastes. The Project will not include septic systems and the sewer system will be designed to current standards which minimizes the potential for leaks. Thus pet wastes are the primary source of concern. The PDFs will include source controls, LID BMPs, and treatment controls which in combination should help to reduce pathogen indicator levels in post-construction stormwater runoff. Pathogens are not expected to occur at elevated levels during the construction-phase of the Project. On this basis, the Projects impact on pathogen and pathogen indicators is considered less than significant.
- ***Hydrocarbons*:** Hydrocarbon concentrations will likely increase in post-development because of vehicular emissions and leaks. In stormwater runoff, hydrocarbons are often associated with soot particles that can combine with other solids in the runoff. Such materials are subject to treatment in the proposed LID BMPs and treatment control PDFs. Source control BMPs incorporated in compliance with the MS4 Permit and the SUSMP requirements will also minimize the presence of hydrocarbons in runoff. During the construction phase of the Project, pursuant to the Construction General Permit, the Construction Stormwater Pollution Prevention Plan must include BMPs that address proper

handling of petroleum products on the construction site, such as proper petroleum product storage and spill response practices, and those BMPs must effectively prevent the release of hydrocarbons to runoff per the Best Available Technology Economically Achievable and Best Conventional Pollutant Control Technology standards. On this basis, the impact of the Project on hydrocarbons is considered less than significant.

- ***Trash and debris*:** Trash and debris in runoff are likely to increase in post-development if left unchecked. However, the Project PDFs, including source control and treatment BMPs incorporated in compliance with the MS4 Permit and the SUSMP requirements, will minimize the adverse impacts of trash and debris. Source controls such as street sweeping, public education, fines for littering, covered trash receptacles, and storm drain stenciling are effective in reducing the amount of trash and debris that is available for mobilization during wet weather. Trash and debris will be captured in catch basin inserts in high use parking lots and in the LID BMP and treatment control PDFs. During the construction phase of the Project, PDFs implemented per Construction General Permit and General De-Watering Permit requirements will remove trash and debris through the use of BMPs such as catch basin inserts and by general good housekeeping practices. Trash and debris are not expected to significantly impact receiving waters due to the implementation of the Project PDFs.
- ***Methylene Blue Activated Substances (MBAS):*** In the post-development phase, the presence of soap in runoff from the Project will be controlled through the source control PDFs, including a public education program on residential and charity car washing and the provision of a centralized car wash area directed to sanitary sewer in the multi-family residential areas. Other sources of MBAS, such as cross connections between sanitary and storm sewers, are unlikely given modern sanitary sewer installation methods and inspection and maintenance practices. During the construction phase of the Project, equipment and vehicle washing will not use soaps or any other MBAS sources. Therefore, MBAS are not expected to significantly impact the receiving waters of the proposed Project.
- ***Cyanide:*** In addition to the expected relatively low level of cyanide in untreated stormwater, cyanide in runoff from the Project would be readily removed by biological uptake, degradation by microorganisms, and by volatilization in the LID BMP and treatment control PDFs. Therefore cyanide is not expected to significantly impact the receiving waters of the Project.
- ***Bioaccumulation*:** In the literature, the primary pollutants that are of concern with regard to bioaccumulation are mercury and selenium. However, selenium and mercury are not of concern in the Project's watershed, so bioaccumulation of selenium and mercury is also not expected to result either during the construction or post-development Project phases. On this basis, the potential for

bioaccumulation in the Project PDFs or in the Santa Clara River and adverse effects on waterfowl and other species is considered less than significant.

- ***Construction Impacts:*** Construction impacts on water quality are generally caused by soil disturbance and subsequent suspended solids discharge. These impacts will be minimized through implementation of construction BMPs that will meet or exceed measures required by the Construction General Permit, as well as BMPs that control the other potential construction-related pollutants (e.g., petroleum hydrocarbons and metals). A SWPPP specifying BMPs for the site that meet or exceed Best Available Technology economically achievable/Best Conventional Pollutant Control Technology standards will be developed as required by, and in compliance with, the Construction General Permit and Los Angeles County Standard Conditions. Erosion control BMPs, including but not limited to hydro-mulch, erosion control blankets, stockpile stabilization, and other physical soil stabilization techniques will be implemented to prevent erosion, whereas sediment controls, including but not limited to silt fencing, sedimentation ponds, and secondary containment on stockpiles will be implemented to trap sediment and prevent discharge. Non-stormwater and construction waste and materials management BMPs, such as vehicle and equipment fueling and washing BMPs; nonvisible pollutant monitoring; and BMPs to manage materials, products, and solid, sanitary, concrete, hazardous, and hydrocarbon wastes will also be deployed to protect construction site runoff quality. On this basis, the construction-related impact of the project on water quality is considered less than significant.
- ***Regulatory Requirements:*** The proposed Project satisfies MS4 Permit requirements for new development, including SUSMP requirements and SQMP requirements, satisfies the LID requirements of the Los Angeles County LID Manual and the Newhall Ranch Sub-Regional Stormwater Mitigation Plan, and satisfies construction-related requirements of the Construction General Permit and General Dewatering Permit, and therefore complies with water quality regulatory requirements applicable to stormwater runoff.

8.2 Groundwater Impacts

- ***Groundwater Quality Impacts (Nitrate-N+Nitrite-N)***: MS4 Permit, Construction General Permit, Dewatering General Permit, and SUSMP-compliant BMPs will be incorporated into the Project to address nutrients in both the construction phase and post-development. Nitrate-nitrogen plus nitrite-nitrogen concentrations are predicted to decrease in the post-developed condition. The predicted nitrate-nitrogen plus nitrite-nitrogen concentration in stormwater runoff after treatment in the Project PDFs and in irrigation water is well below the groundwater quality objective. On this basis, the potential for adversely affecting groundwater quality is considered less than significant.

- ***Groundwater Recharge Impacts***: Project stormwater runoff will be discharged to the Santa Clara River after treatment, whose channel is predominantly natural and consists of vegetation and coarse-grained sediments (rather than concrete). The porous nature of the sands and gravels forming the streambed will allow for significant infiltration to occur to the underlying groundwater. Also, irrigation water is predicted to be fully infiltrated during dry weather, which will increase groundwater recharge from the Project. On this basis, the Project's impact on groundwater recharge is considered less than significant

8.3 Hydromodification Impacts

The following are the conclusions regarding the significance of impacts for hydromodification impacts under wet- and dry-weather conditions:

- ***Wet Weather Project Impacts***: Although the Project's runoff volumes, flow rates, and durations will increase, potential impacts of hydromodification (i.e., the potential to cause erosion, siltation, or channel instability) will be avoided, minimized, and mitigated by the Project PDFs in the following ways:
    - Project site design PDFs, especially open space retention and efficient irrigation will avoid and/or minimize increases in runoff volume from the development area, the preferred method for controlling hydromodification impacts from new development (SCCWRP, 2005a).
    - Concentrated flows will be mitigated with energy dissipaters at the discharge points to the Santa Clara River and the Santa Clara River banks will be protected by geomorphically-referenced engineering techniques, primarily with vegetated buried bank stabilization in non-jurisdictional upland areas adjacent to the river. This type of stabilization technique is the preferred approach for bank stabilization (SCCWRP, 2005a).

  For these reasons, direct hydromodification impacts of the Project on the Santa Clara River is considered less than significant.

- ***Cumulative Hydromodification Impacts***: The Project contributes only 0.5 percent of the total potential impervious surface at build out within the watershed, the Project includes hydromodification controls as Project Design Features, future development projects within the watershed will control flow in compliance with the sub-regional program, and large-scale changes naturally occur in the Santa Clara River in response to major episodic events, therefore, the Project's contribution to cumulative hydromodification impacts to the Santa Clara River will be less than significant and consistent with the requirements of the MS4 permit.

- ***Dry Weather Hydromodification Impacts***: It is predicted that all dry weather flows will be removed in the treatment control PDFs, which also provide hydrologic source control. As a result, no appreciable change in seasonality of flows is anticipated to result from development. Based on the comprehensive site planning, source control, LID, and treatment control strategy and that no dry weather flows are predicted to be discharges to the Santa Clara River, the impact of the Project on dry weather water quality and seasonality of flow in the Santa Clara River is considered less than significant.

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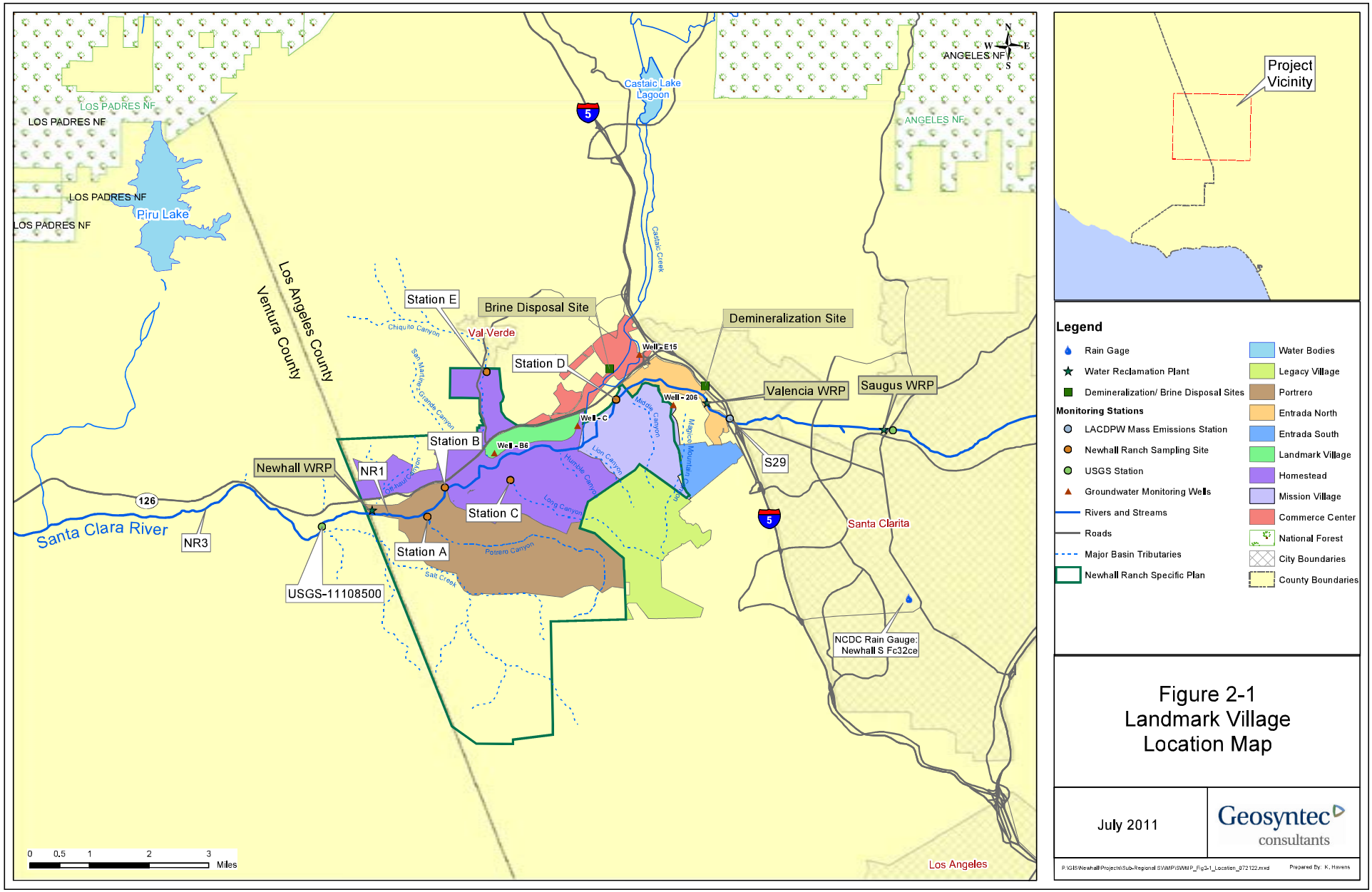

Figure 2-1
Landmark Village
Location Map

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Prepared By: K. Havens

Legend

- Landmark Project Boundary

Model Land Use

- Single-Family
- Multi-Family
- Commercial
- Recreation
- School
- Park
- Open Space
- Water Quality Facility
- Road
- Off-site Bridge
- Off-site SR-126 Right of Way

Not shown: Off-site Water Tanks

Figure 2-2: Landmark Village Project Land Uses

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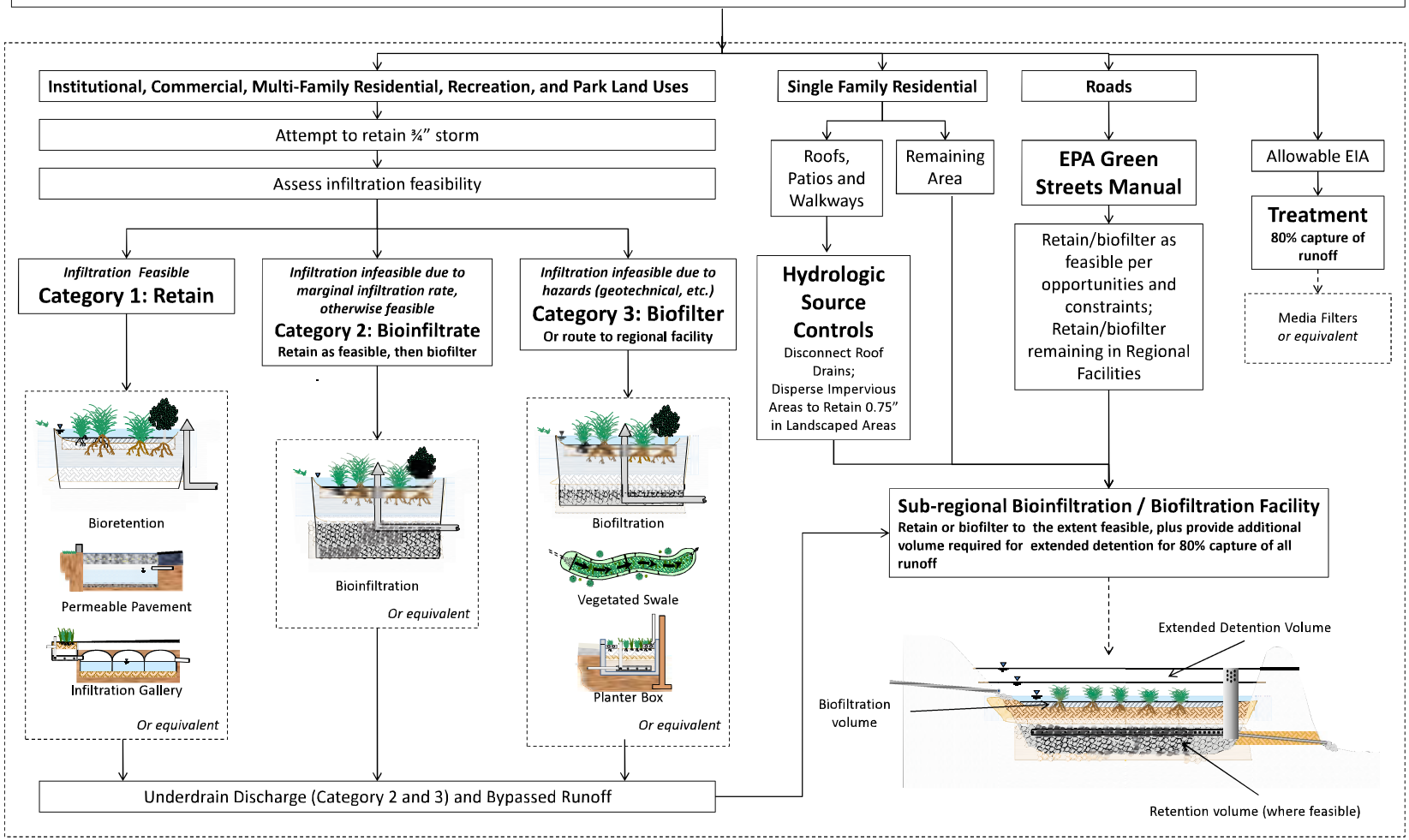
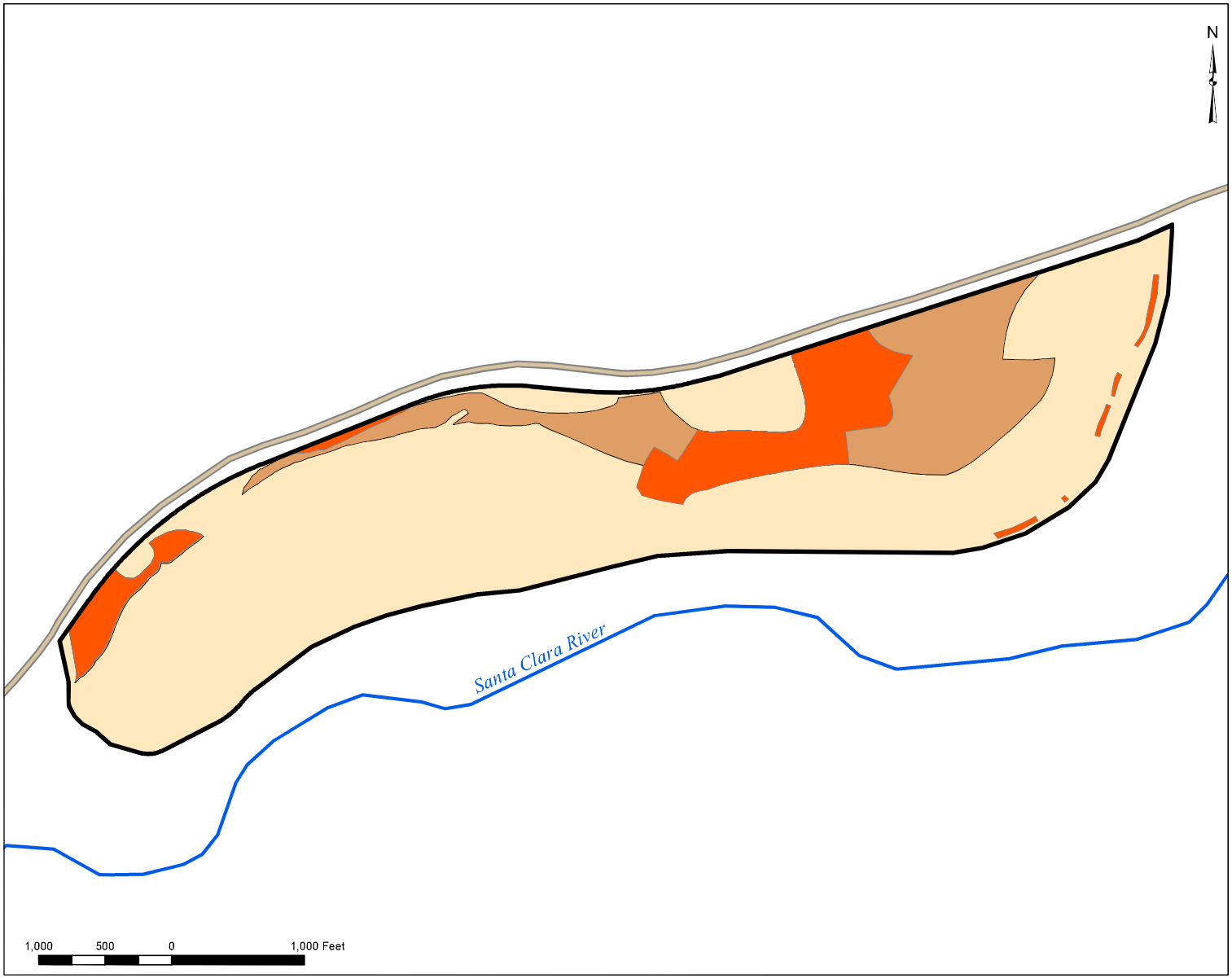
Prepared By: K. Havens

LANDMARK VILLAGE LID PERFORMANCE STANDARD

LID project design features (PDFs) shall be selected and sized to retain the volume of stormwater runoff produced from a 0.75 inch storm event to reduce the percentage of Effective Impervious Area (EIA) to 5 percent or less of the total project area within the vesting tentative map project and associated off-site project area. Runoff from all EIA shall be treated with treatment control measures that are selected to address the pollutants of concern and are sized to capture and treat 80 percent of the average annual runoff volume.



Landmark Village LID Performance Standard Landmark Village LID Water Quality Assessment	Geosyntec consultants	**Figure 5-1**



Legend

- Landmark Project Boundary
- Infiltration Feasible*
- Infiltration Feasible**
- Partial Infiltration Feasible

* Infiltration likely feasible directly from BMP bottom

** Infiltration likely feasible with supplemental dry well

Figure 5-2:
Landmark Village
Regional Facilities and
Infiltration Feasibility Constraints

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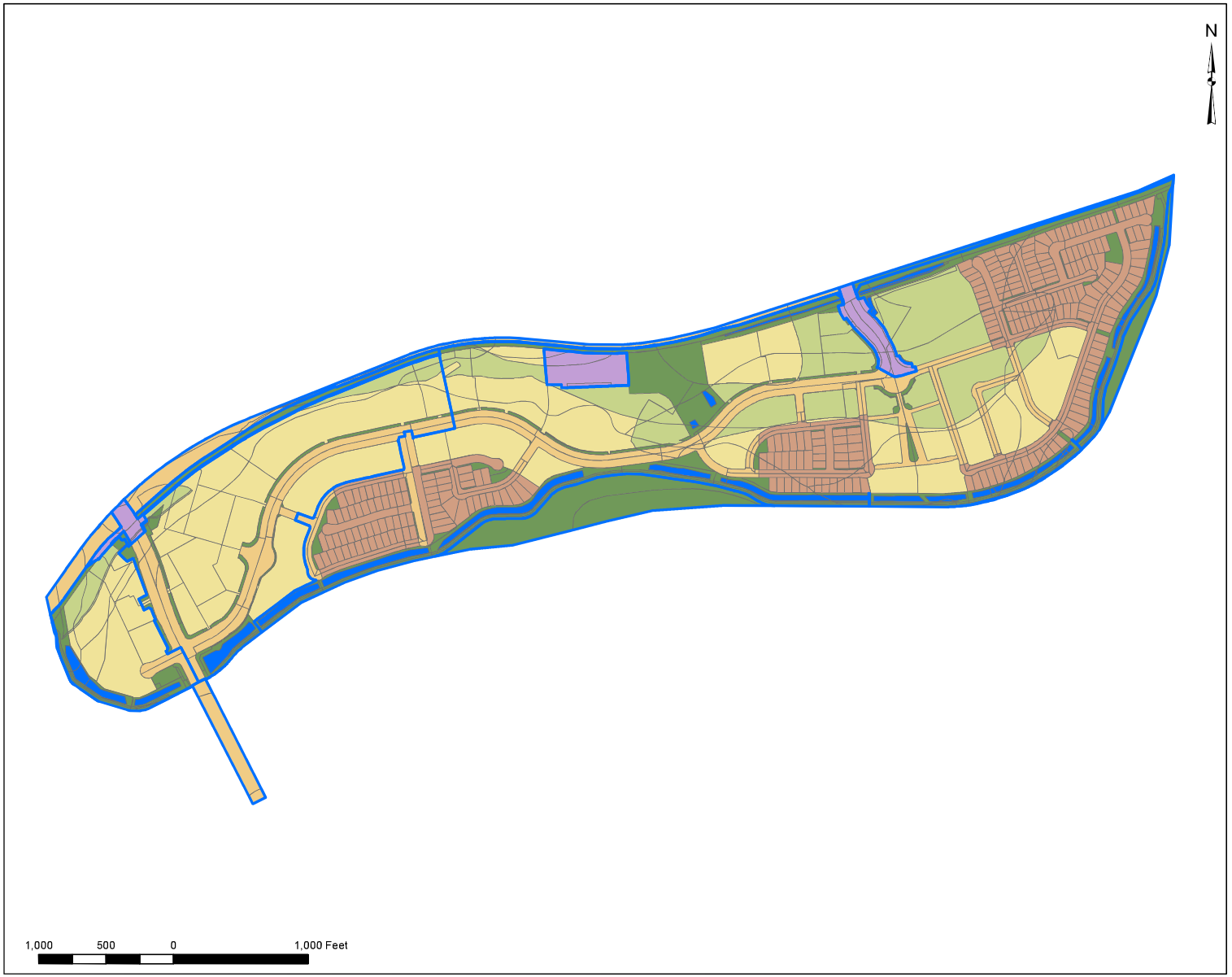
Prepared By: K. Havens

Note: Source: Alan E. Seward, 2010.



Legend

- Sub-regional and Treatment BMP Drainage Areas

Parcel-based Treatment Category:

- Open Space
- Category 1
- Category 2
- Green Streets
- Single-Family
- Treatment BMP
- Sub-regional Facility

Figure 5-3: Landmark Village Distributed BMP Model Categories

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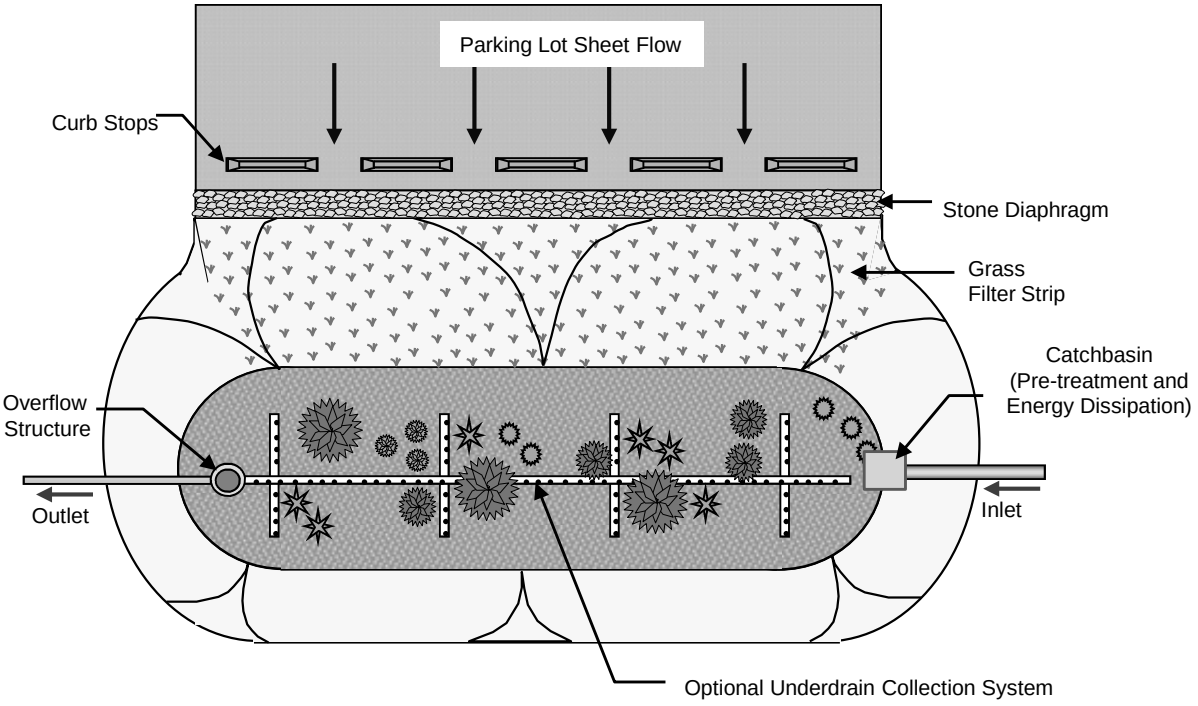
Prepared By: K. Havens

Plan View



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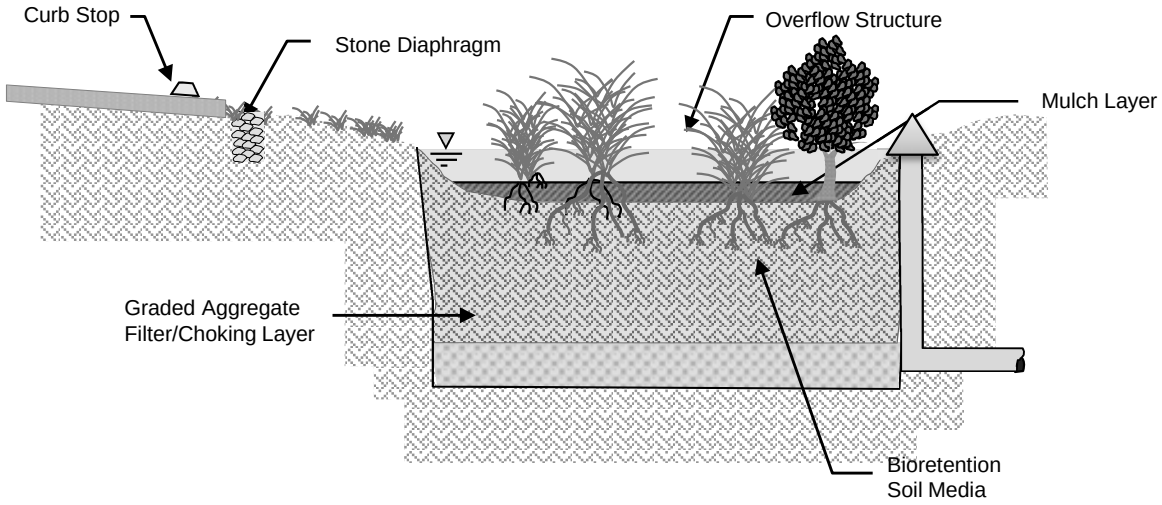

Conceptual Illustration of a Bioretention Facility

Landmark Village

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Figure 5-4

Oakland | July 2011

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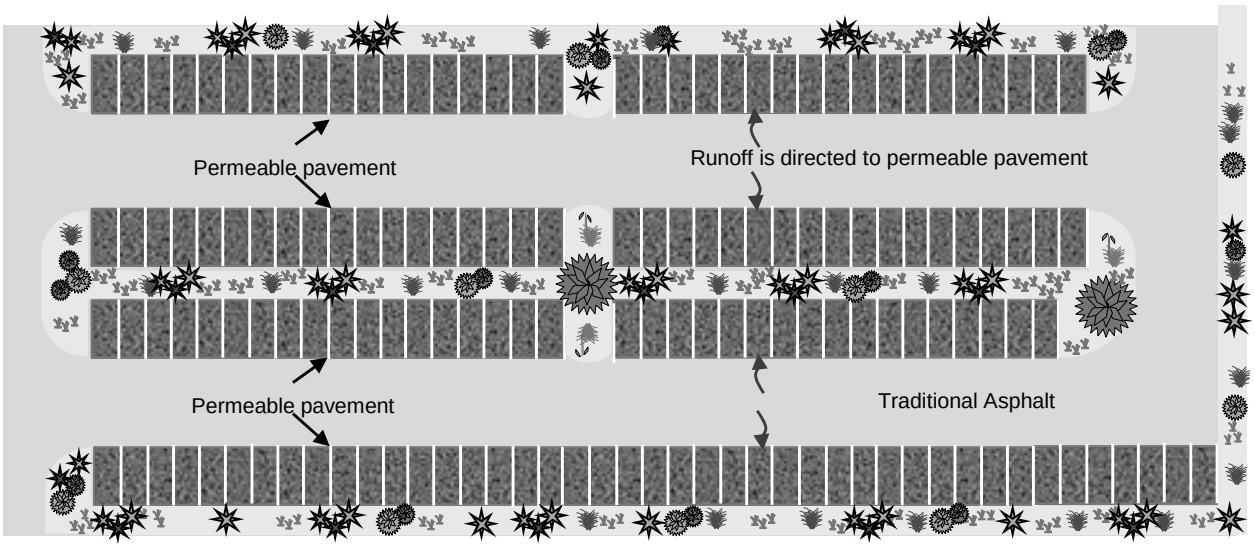
Plan View



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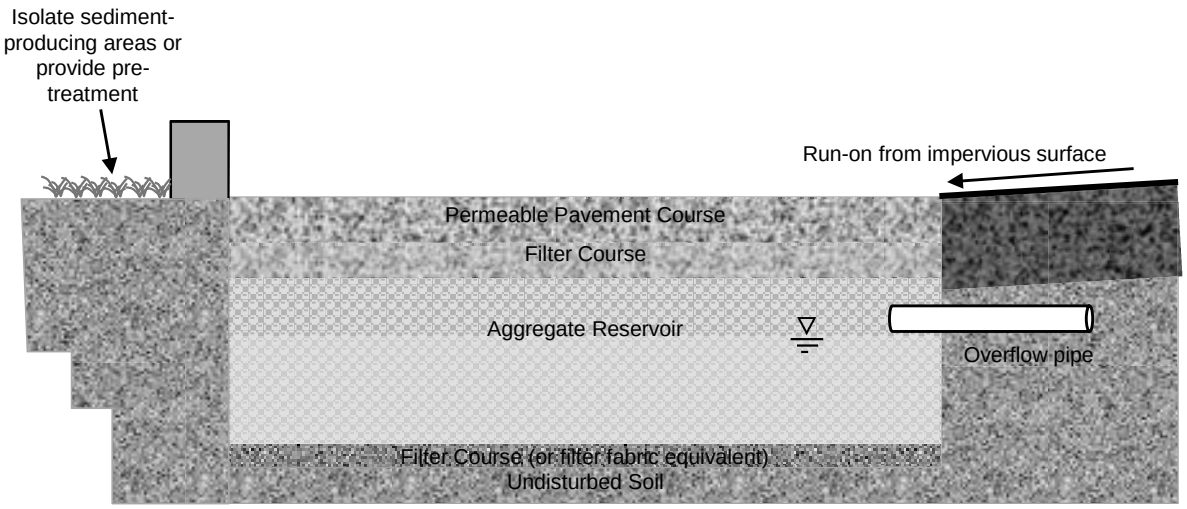

Conceptual Illustration of Permeable Pavement

Landmark Village

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Figure 5-5

Oakland | July 2011

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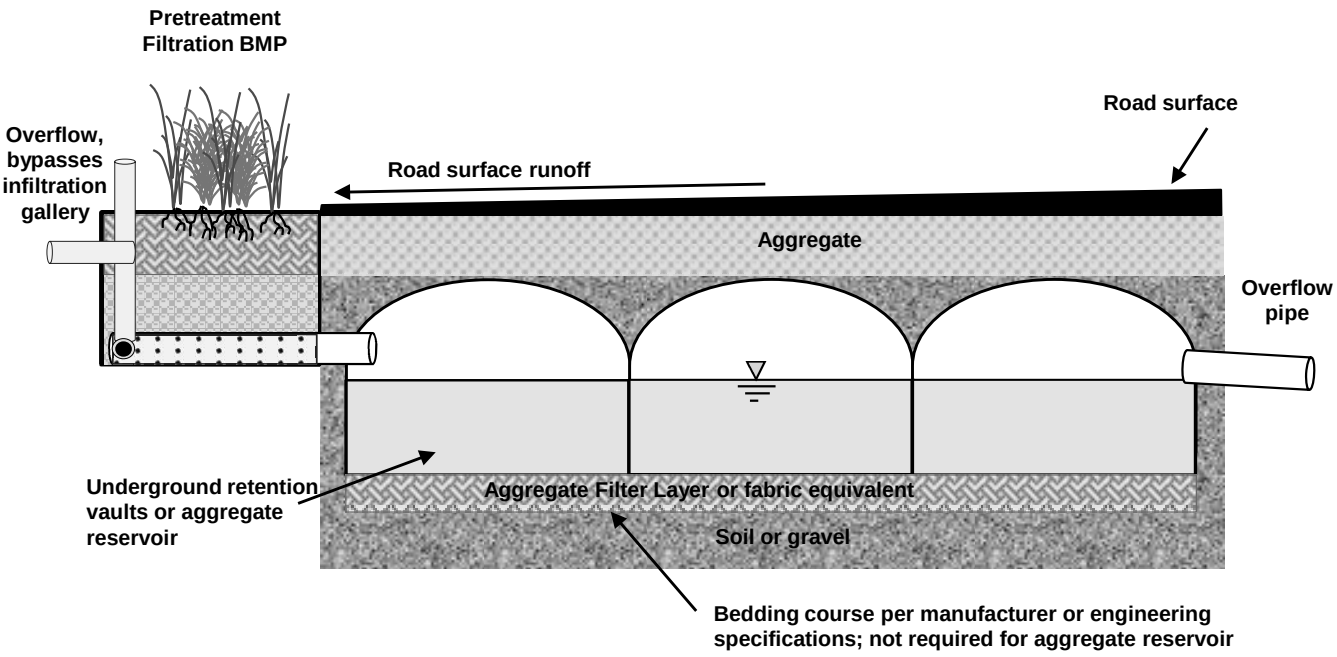

Conceptual Illustration of an Infiltration Gallery

Landmark Village

Geosyntec consultants

Figure 5-6

Oakland | July 2011

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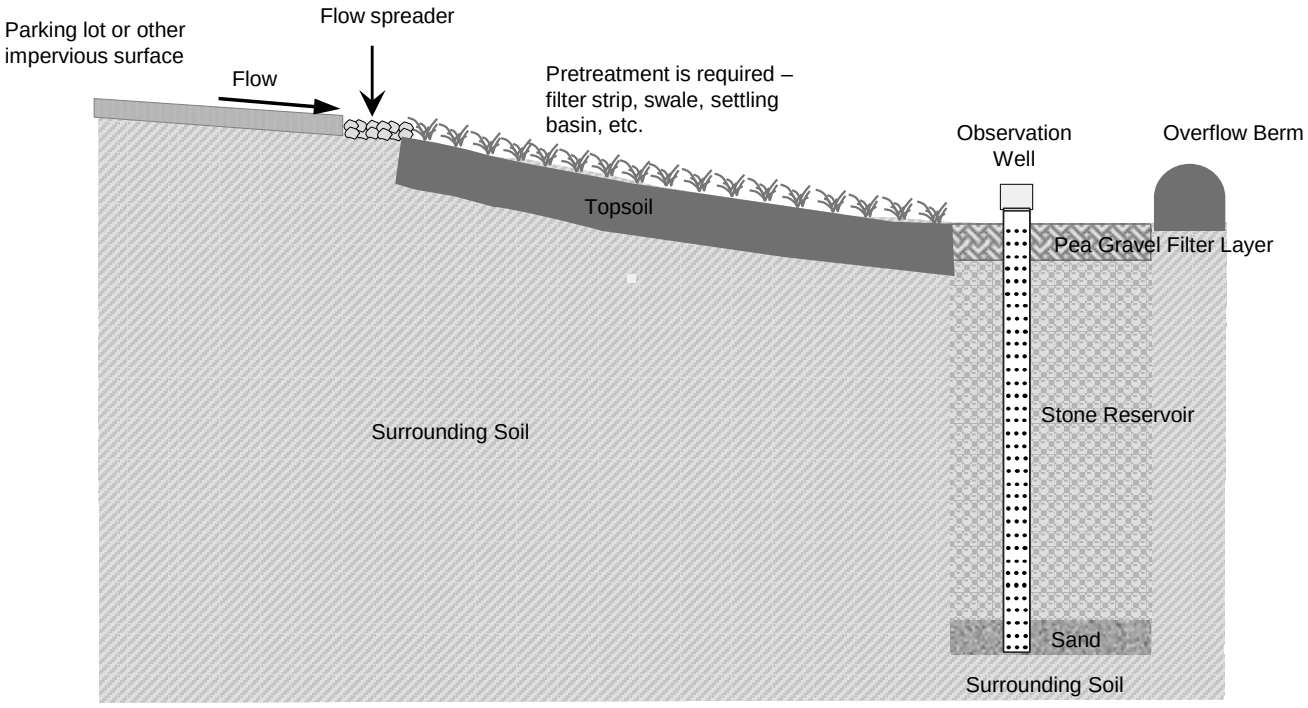

Conceptual Illustration of an Infiltration Trench

Landmark Village

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Figure 5-7

Oakland | July 2011

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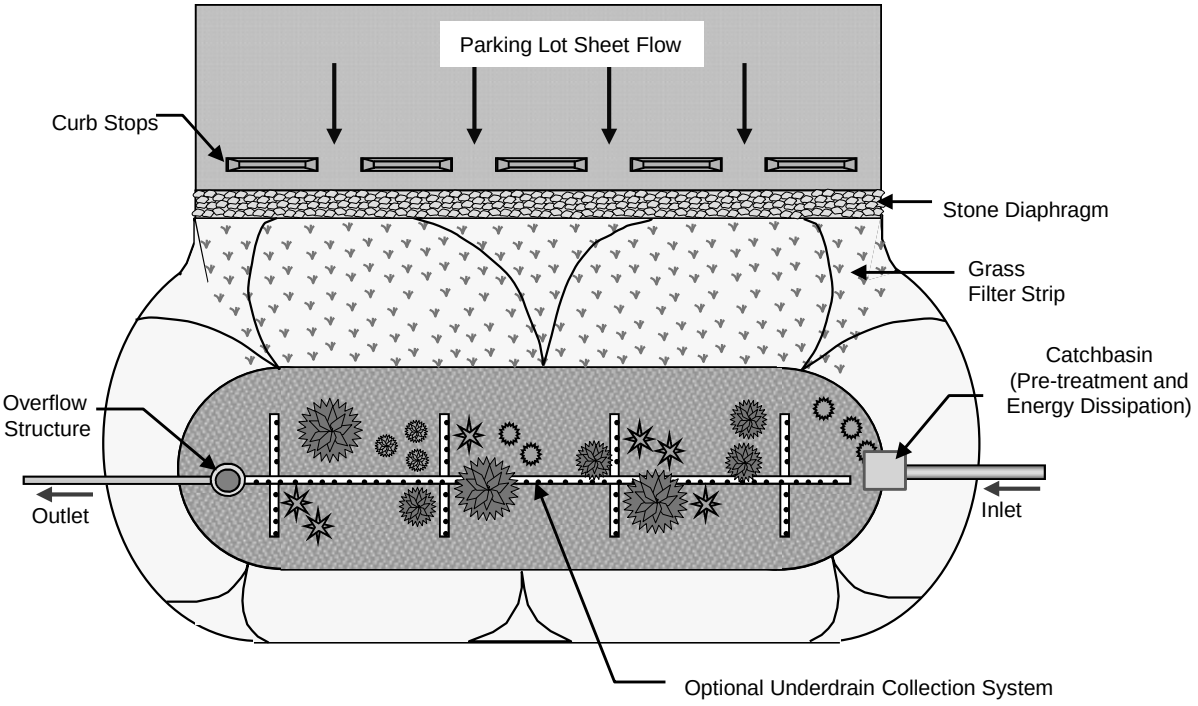
Plan View



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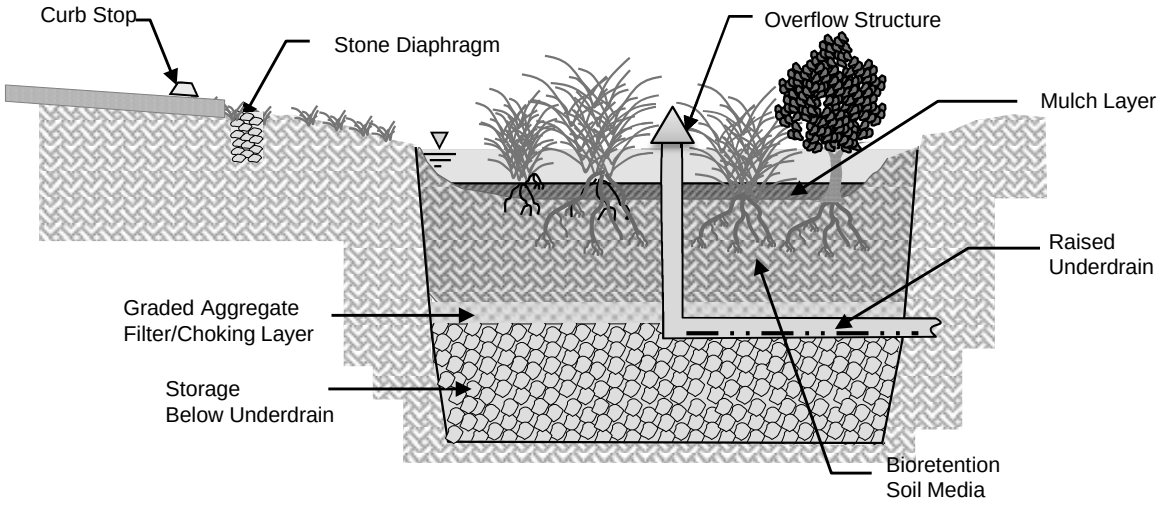

Conceptual Illustration of a Bioinfiltration Facility

Landmark Village

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Figure 5-8

Oakland | July 2011

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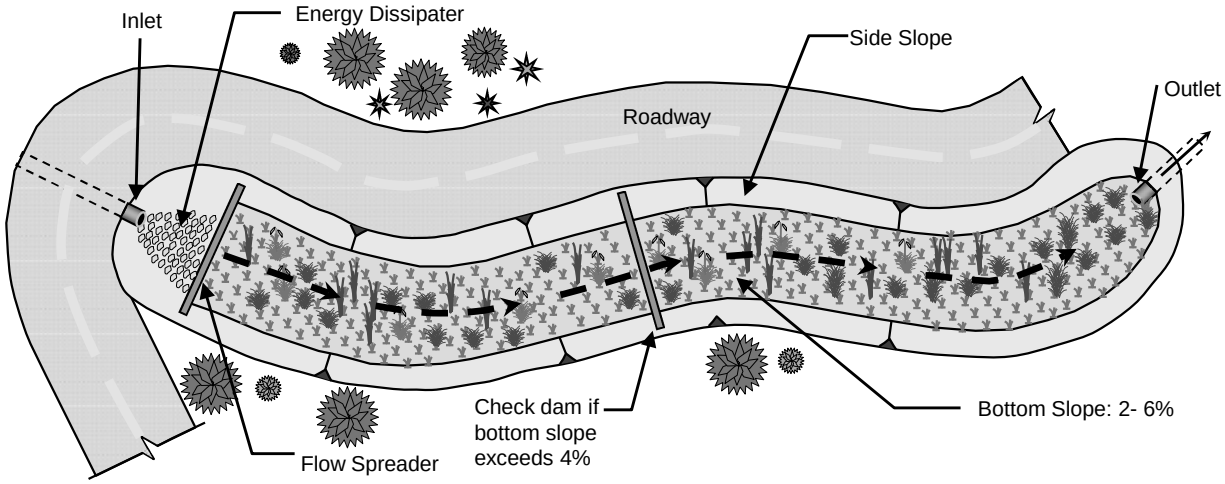
Plan View



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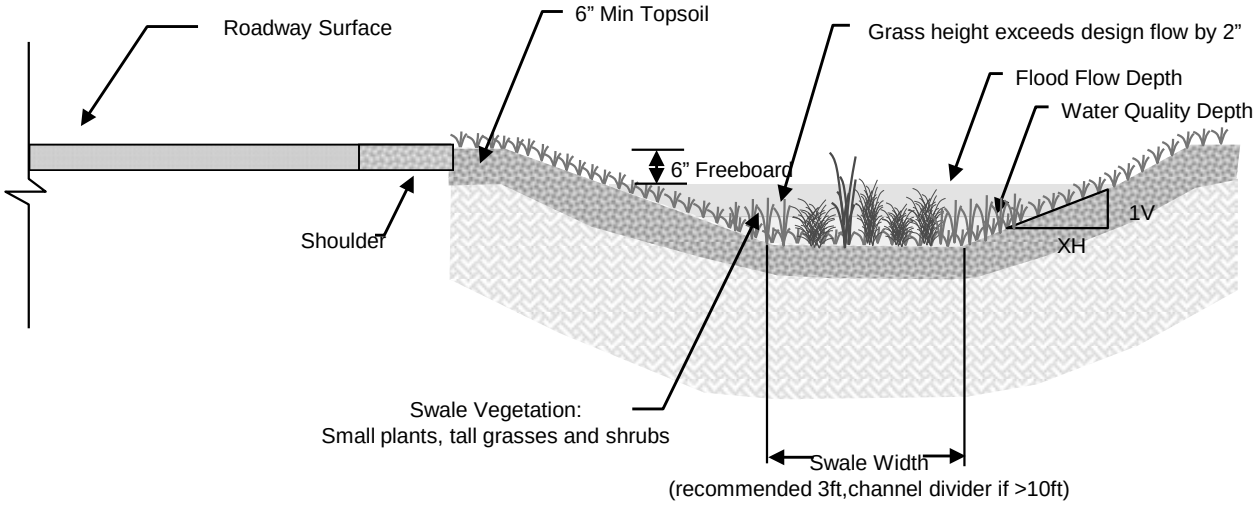

Conceptual Illustration of a Vegetated Swale

Landmark Village

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Figure 5-9

Oakland | July 2011

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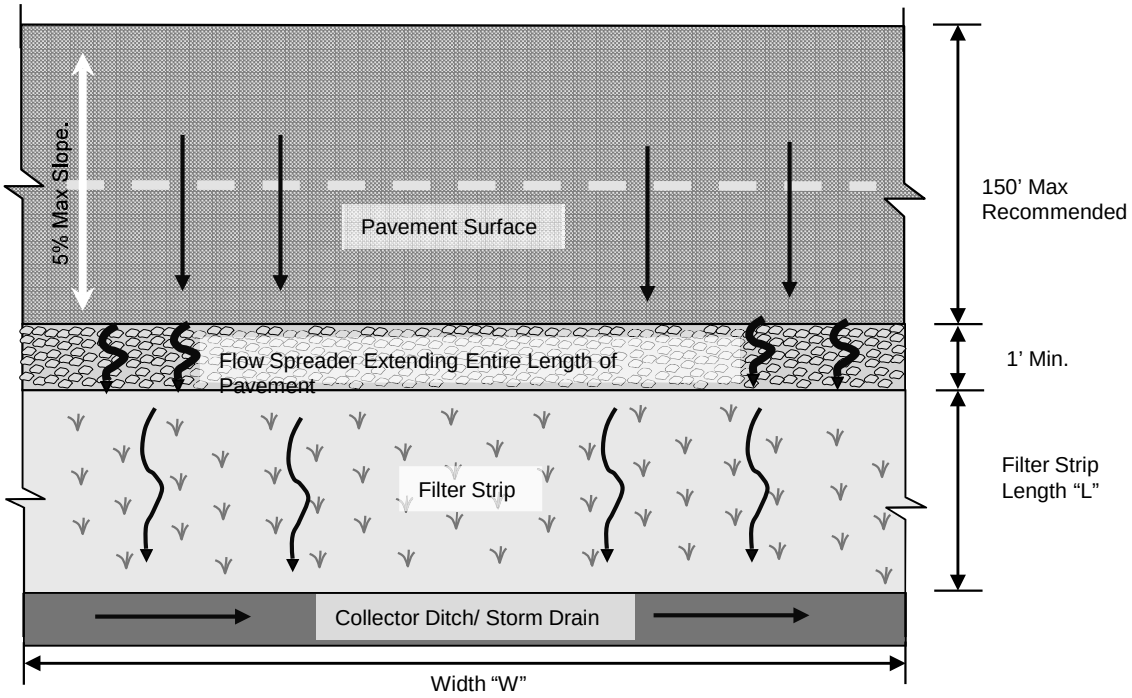
Plan View



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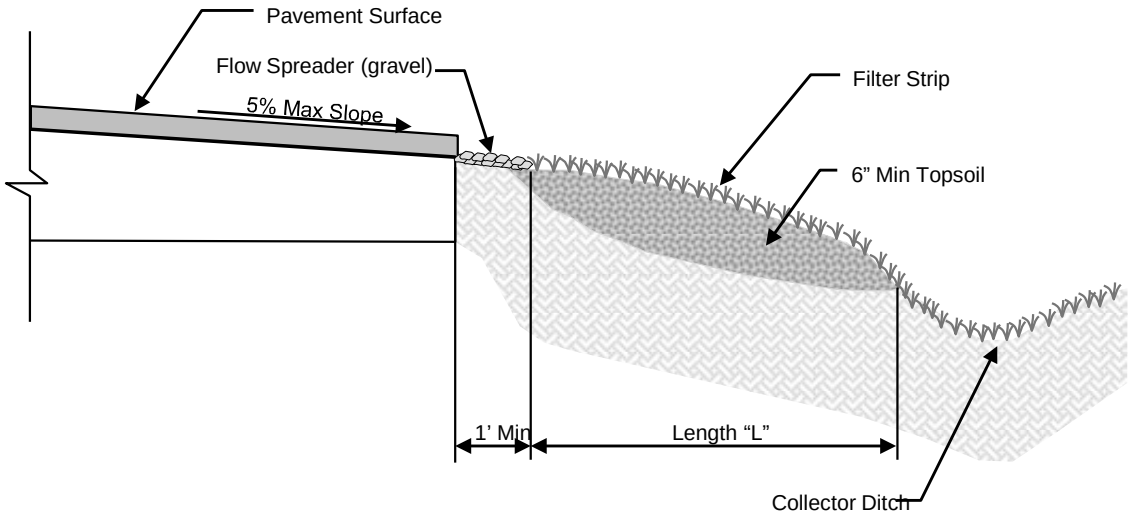

Conceptual Illustration of a Vegetated Filter Strip

Landmark Village

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Figure 5-10

Oakland | July 2011

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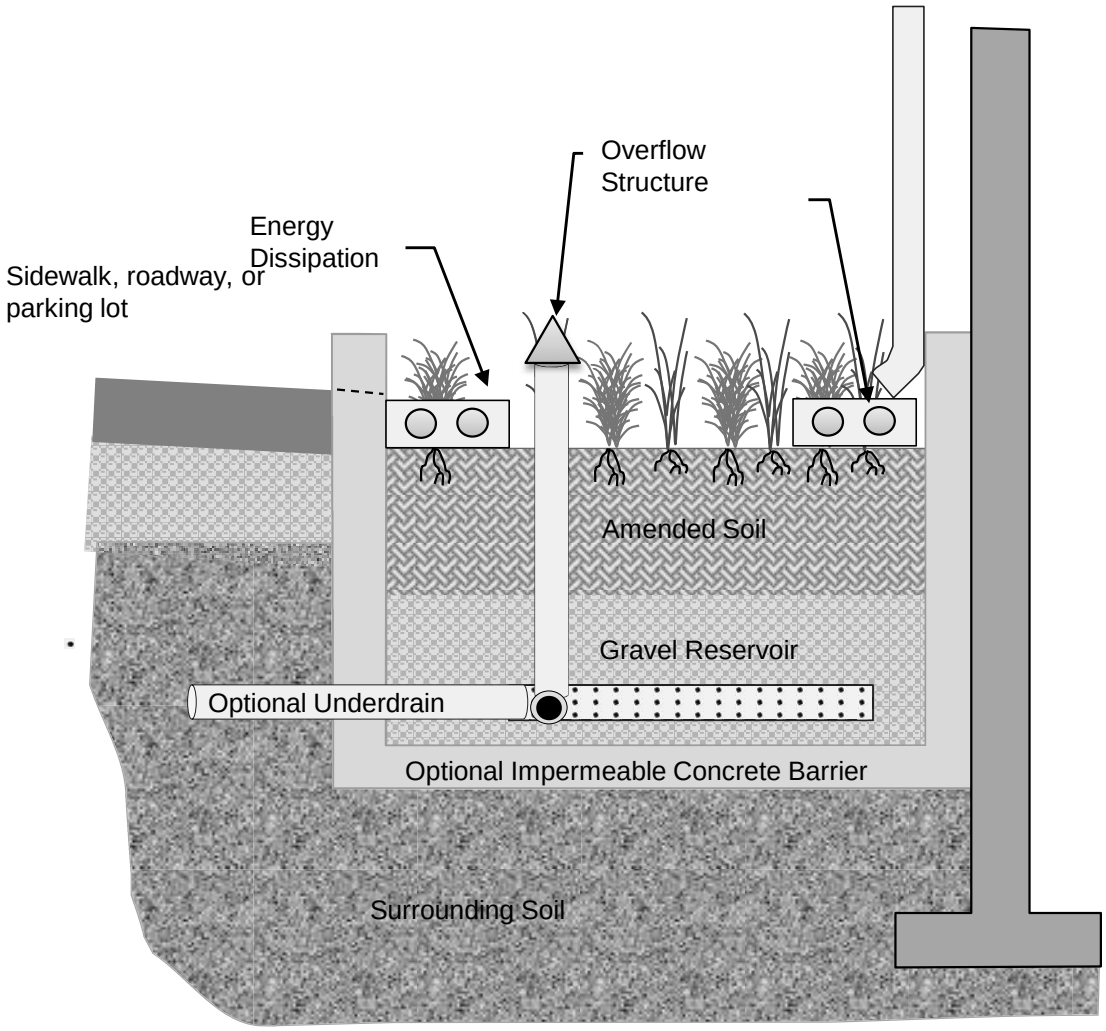

Conceptual Illustration of a Planter Box

Landmark Village

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Figure 5-11

Oakland | July 2011

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Examples of Roof Dispersion

Landmark Village

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Figure 5-12

Oakland | July 2011

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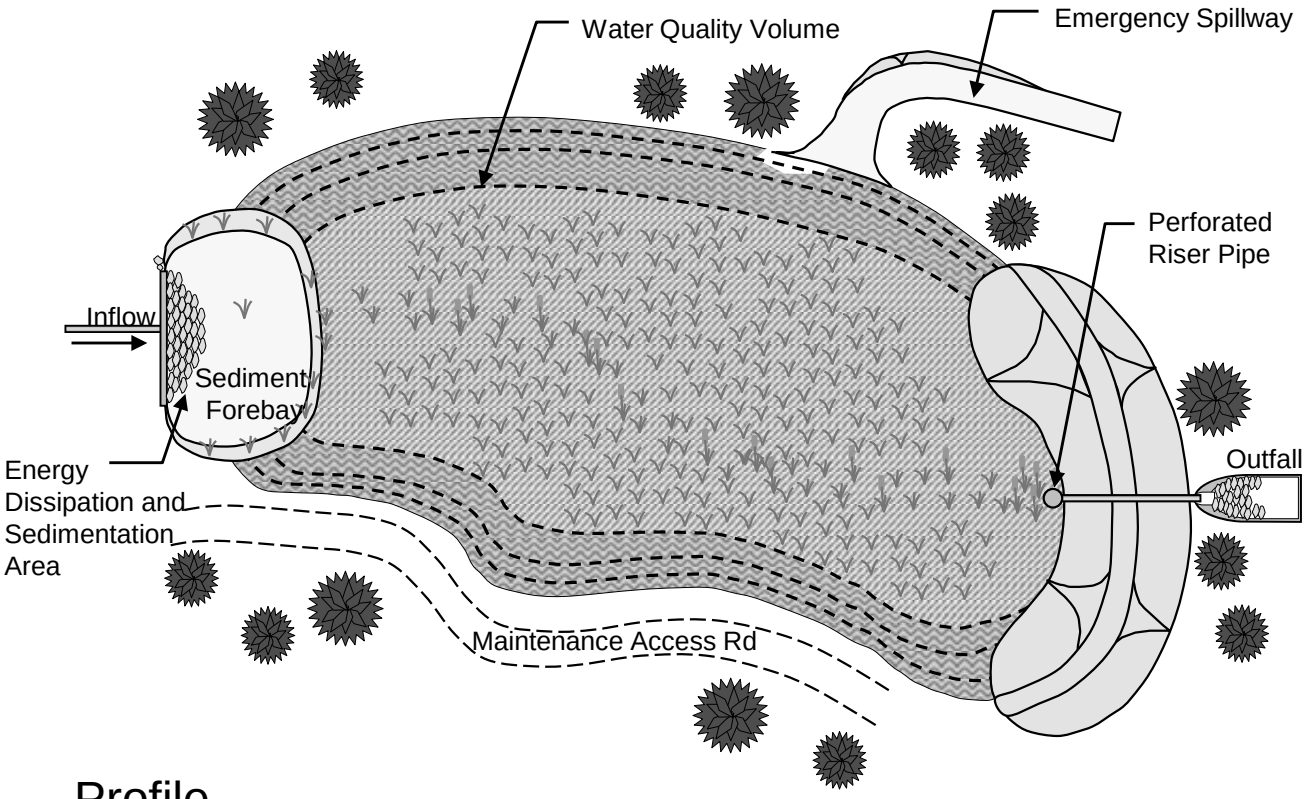
Plan View



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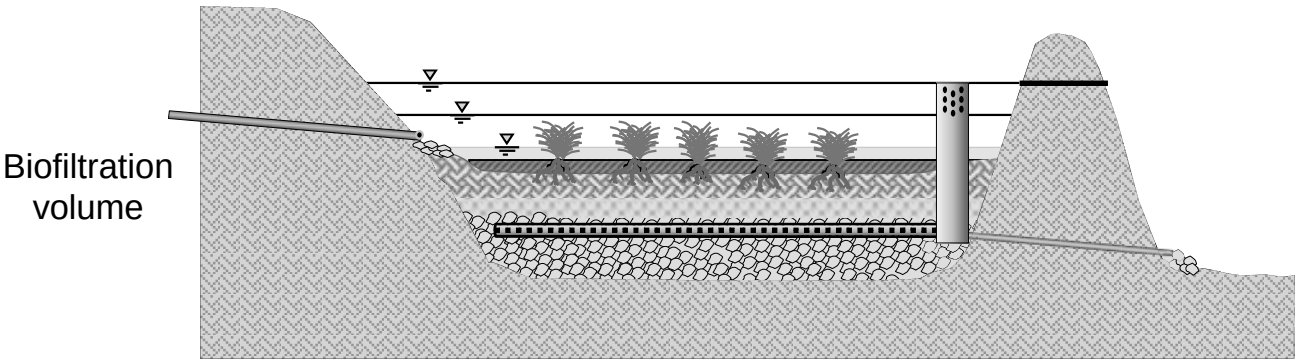

Conceptual Illustration of a Regional Infiltration/Biofiltration Facility
Landmark Village

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Figure 5-13

Oakland | July 2011

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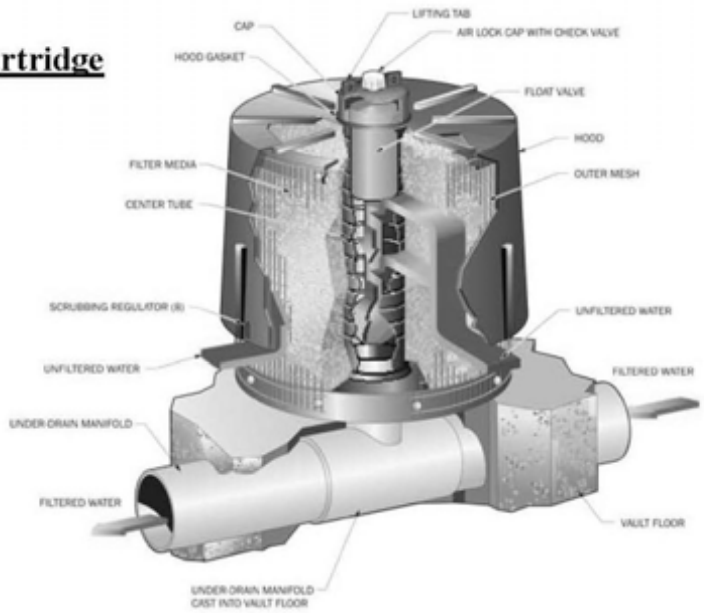
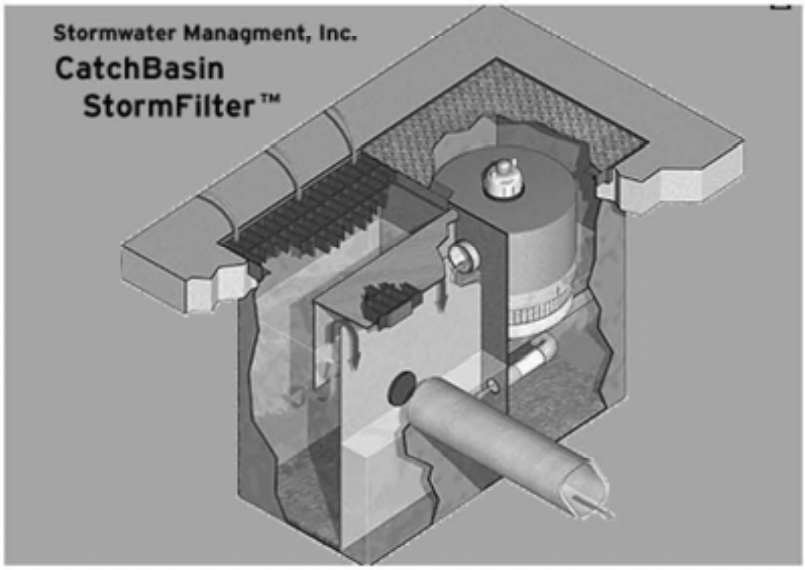
StormFilter Media Cartridge



Example configuration of Catch Basin Storm Filter



Conceptual Illustration of a Media Filter

Landmark Village

Geosyntec consultants

Figure 5-14

Oakland | July 2011

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Landmark Village Water Quality Technical Report Appendices

(A) Pollutants of Concern

(B) Modeling Parameters & Methodology

(C) Newhall Ranch Stormwater Monitoring Data

(D) "Review of Bacteria Data from Southern California Watersheds"

(E) "Effect of Urbanization on Groundwater Recharge in the Santa Clarita Valley"

(F) "Assessment of Potential Impacts Resulting from Cumulative Hydromodification Effects, Selected Reaches of the Santa Clara River, Los Angeles County, California"

(G) LID Water Quality Modeling Methodology

(H) Geologic/Geotechnical Report, Assessment of Infiltration Parameters

A. POLLUTANTS OF CONCERN

A.1. Pollutants of Concern

Pollutant of Concern [(1)]	Rationale for Selection / Exclusion	Significance Criteria
Sediment: Total Suspended Solids (TSS) & Turbidity	1. Sediment is a common component of stormwater, and can be a pollutant. Sediment can be detrimental to aquatic life (primary producers, benthic invertebrates, and fish) by interfering with photosynthesis, respiration, growth, reproduction, and oxygen exchange in water bodies. Sediment can transport other pollutants that are attached to it including nutrients, trace metals, and hydrocarbons. Sediment is the primary component of total suspended solids (TSS), a common water quality analytical parameter (CASQA, 2003). 2. Turbidity is a measure of suspended matter that interferes with the passage of light through the water or in which visual depth is restricted. Turbidity may be caused by a wide variety of suspended materials, which range in size from colloidal to coarse dispersions, depending upon the degree of turbulence. In lakes or other waters existing under relatively quiescent conditions, most of the turbidity will be due to colloidal and extremely fine dispersions. In rivers under flood conditions, most of the turbidity will be due to relatively coarse dispersions. Erosion of clay and silt soils may contribute to in-stream turbidity. Organic materials reaching rivers serve as food for bacteria, and the resulting bacterial growth and other microorganisms that feed upon the bacteria produce additional turbidity. Nutrients in runoff may stimulate the growth of algae, which may also contribute to turbidity. Discharges of turbid runoff are primarily of concern during the construction phase of development.	1. Narrative objective in the Basin Plan: "Water shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses." 2. Basin Plan objective for turbidity: "Waters shall be free of changes in turbidity that cause nuisance or adversely affect beneficial uses. Increases in natural turbidity attributable to controllable water quality factors shall not exceed the following limits: Natural Turbidity — Max Increase 0-50 NTU — 20% > 50 NTU — 10% Allowable zones of dilution within which higher concentrations may be tolerated may be defined for each discharge in specific Water Discharge Requirements."

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Nutrients: Ammonia, Nitrite, Nitrate, Total Nitrogen, and Total Phosphorus	1. Nutrients including nitrogen and phosphorus are the major plant nutrients used for fertilizing landscapes, and are often found in stormwater. These nutrients can result in excessive or accelerated growth of vegetation, such as algae, resulting in impaired use of water in lakes and other sources of water supply. For example, nutrients have led to a loss of water clarity in Lake Tahoe. In addition, un-ionized ammonia (one of the nitrogen forms) can be toxic to fish (CASQA, 2003).	1. Basin Plan standards for ammonia: "In order to protect aquatic life, ammonia concentrations in receiving waters shall not exceed the values listed for the corresponding in-stream conditions in Tables 3-1 to 3-4." The criterion for ammonia in Tables 3-1 to 3-4 varies with pH and temperature; the criterion is lower for lower pH and temperature. The basin plan amendment for updated ammonia standards (dated 04/02, effective July 15, 2003) will be used. 2. Basin Plan surface water standards for nitrogen: "Waters shall not exceed 10 mg/L nitrogen as nitrate-nitrogen plus nitrite-nitrogen (NO_3-N + NO_2-N), 45 mg/L as nitrate (NO_3), 10 mg/L as nitrate-nitrogen (NO_3-N), or 1 mg/L as nitrite-nitrogen (NO_2-N) or as otherwise designated in Table 3-8." Table 3-8 lists Santa Clara River Reach 5 with a water quality objective of 5 mg/L nitrate-N + nitrite-N. 3. Basin Plan groundwater standards for nitrogen: "Ground waters shall not exceed 10 mg/L nitrogen as nitrate-nitrogen plus nitrite-nitrogen (NO_3-N + NO_2-N), 45 mg/L as nitrate (NO_3), 10 mg/L as nitrate-nitrogen (NO_3-N), or 1 mg/L as nitrite-nitrogen (NO_2-N)." 4. Resolution 03-011 (LARWQCB, 08/2003) promulgates Nitrogen Compounds TMDLs for Santa Clara River Reach 5. The numeric target for NO3-N + NO2-N in the Nitrogen Compounds TMDL was based on achieving the existing water quality objective of 5 mg/L NO3-N + NO2-N. The numeric target that was used to calculate the wasteload allocations included a 10% margin of safety; thus the numeric target is 4.5 mg/L NO3-N + NO2-N (30-day average). The water quality objectives for ammonia in Reach 5 used in the Nitrogen Compounds TMDL are:

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
		TMDL Ammonia Water Quality Objective (mg/L as N) 1-hr average / 30-day average Reach 5 at County Line 3.4 / 1.2 Reach 5 below Valencia 5.5 / 2.0 Reach 5 above Valencia 4.8 / 2.0 5. Narrative objective for biostimulatory substances in the Basin Plan: "Waters shall not contain biostimulatory substances in concentrations that promote algal growth to the extent that such growth causes nuisance or adversely affects beneficial uses."
Trace metals: Aluminum, Copper, Lead, and Zinc	1. Trace metals are commonly found in stormwater. Many of the artificial surfaces of the urban environment (e.g., galvanized metal, paint, automobiles, or preserved wood) contain metals, which enter stormwater as the surfaces corrode, flake, dissolve, decay, or leach. Over half the trace metal load carried in stormwater is associated with sediments. Metals are of concern because they can be toxic to aquatic organisms, can bioaccumulate (accumulate to toxic levels in aquatic animals such as fish), and have the potential to contaminate drinking water supplies (CASQA, 2003). 2. Aluminum has been identified by the DPW as a constituent of concern for the Santa Clara River based on monitoring conducted at mass emission station S29 (LACDPW, 2005).	1. Narrative objective in the Basin Plan: "All waters shall be maintained free of toxic substances in concentrations that are toxic to, or that produce detrimental physiological responses in human, plant, animal, or aquatic life. …" 2. The CTR criteria are the applicable water quality objectives for protection of aquatic life (40 CFR §131.38). The CTR criteria are expressed for acute and chronic (4-day average) conditions; however, only acute conditions are applicable for stormwater discharges because the duration of stormwater discharge is typically less than 4 days in the Project area. 3. CTR criteria are determined on the basis of hardness in the receiving water. In application of criteria to the Landmark Village project, a hardness value of 250 mg/L based on the minimum observed value at USGS monitoring station will be used.

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
		4. CTR criteria at 250 mg/L hardness are as follows: a. Dissolved copper – 32 µg/L. b. Total lead – 260 µg/L. c. Dissolved zinc – 250 µg/L. 5. The CTR does not include aluminum. The NAWQC contains an acute criterion for acid soluble aluminum (750 µg/L for a pH range of 6.5 to 9.0).
Chloride	1. Resolution No. R03-008, Amendment to the Water Quality Control Plan (Basin Plan) for the Los Angeles Region to Incorporate a Total Maximum Daily Load for Chloride in the Upper Santa Clara River (07/03) states: Elevated chloride concentrations are causing impairments of the water quality objective in Reach 5 and Reach 6 of the Santa Clara River. This objective was set to protect all beneficial uses; agricultural beneficial uses have been determined to be most sensitive, and not currently attained at the downstream end of Reach 5 and Reach 6 in the Upper Santa Clara River. Irrigation of salt sensitive crops such as avocados and strawberries with water containing elevated levels of chloride results in reduced crop yields. Chloride levels in groundwater are also rising.	1. The Basin Plan chloride objective for Reach 5 of the Santa Clara River is 100 mg/L. 2. The TMDL wasteload allocation for MS4 discharges into Santa Clara River Reach 5 is 100 mg/L.

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Pathogens (Bacteria, Viruses, and Protozoa)	1. Bacteria and viruses are common contaminants of stormwater. For separate storm drain systems, sources of these contaminants include animal excrement and sanitary sewer overflow. High levels of indicator bacteria in stormwater have led to the closure of beaches, lakes, and rivers to contact recreation such as swimming (CASQA, 2003). 2. Fecal and total coliform are frequently monitored indicator organisms of pathogens. 3. Human-related activities can increase coliform concentrations. 4. Concentrations of coliform in stormwater also can be elevated due to the presence of coliform bacteria from natural sources.	1. Basin Plan objectives are based on the designated uses of the water body. Santa Clara River Reach 5 is listed with a REC1 beneficial use. Resolution No. 01-018 (LARWQCB, 2001) amended the Basin Plan objectives for bacteria in waters with a contact recreation beneficial use. These standards for freshwaters are <u>Geometric Mean</u> <u>Single Sample</u> E. coli ≤ 126/100 mL ≤ 235/100 mL fecal coliform ≤ 200/100 mL ≤ 400/100 mL
Petroleum Hydrocarbons: Oil & Grease and Polycyclic Aromatic Hydrocarbons (PAHs)	1. Oil and grease includes a wide array of hydrocarbon compounds, some of which are toxic to aquatic organisms at low concentrations. Sources of oil and grease include leakage, spills, cleaning and sloughing associated with vehicle and equipment engines and suspensions, leaking and breaks in hydraulic systems, restaurants, and waste oil disposal (CASQA, 2003). 2. Hydrocarbons are hydrophobic (low solubility in water), have the potential to volatilize, and most forms are biodegradable. A subset of hydrocarbons, Polynuclear Aromatic Hydrocarbons (PAHs) can be toxic depending on the concentration levels, exposure history, and sensitivity of the receptor organisms. Of particular concern are those PAH compounds associated with transportation-related sources. 3. Petroleum hydrocarbons are ubiquitous, and used in a wide variety of applications. Potential sources are generally expected to increase with urban development and potentially during construction of the Project.	1. Narrative objective in the Basin Plan for oil & grease: "Waters shall not contain oils, greases, waxes, or other materials in concentrations that result in a visible film or coating on the surface of the water or on objects in the water, that cause nuisance or that otherwise adversely affect beneficial uses." 2. PAHs are a class of compounds. CTR values for individual PAHs are available for protection of human health only. There are no regulatory standards for PAHs for the protection of aquatic health.

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Pesticides	1. Pesticides (including herbicides, fungicides, rodenticides, and insecticides) have been repeatedly detected in stormwater at toxic levels, even when pesticides have been applied in accordance with label instructions. As pesticide use has increased, so too have concerns about adverse effects of pesticides on the environment and human health. Accumulation of these compounds in simple aquatic organisms, such as plankton, provides an avenue for biomagnification through the food web, potentially resulting in elevated levels of toxins in organisms that feed on them, such as fish and birds (CASQA, 2003). 2. Pesticides loads may be present in runoff from developed areas due to pesticide use for urban landscaping.	1. Narrative objective in the Basin Plan: "Waters designated for use as domestic or municipal supply (MUN) shall not contain concentrations of pesticides in excess of the limiting concentrations specified in … Title 22 of the California Code of Regulations …." Title 22 contains maximum contaminant levels for a range of pesticides. 2. CTR lists numeric objectives for some, but not all pesticides. There are no CTR criteria for diazinon and chlorpyrifos, but these pesticides, along with other toxic legacy pesticides such as Chlordane, Dieldrin, DDT, and Toxaphene, are now banned from most residential uses.

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Trash and Debris	1. Gross Pollutants (trash, debris, and floatables) may include heavy metals, pesticides, and bacteria in stormwater. Typically resulting from an urban environment, industrial sites and construction sites, trash and floatables may create an aesthetic "eye sore" in waterways. Gross pollutants also include plant debris (such as leaves and lawn-clippings from landscape maintenance), animal excrement, street litter, and other organic matter. Such substances may harbor bacteria, viruses, vectors, and depress the dissolved oxygen levels in streams, lakes, and estuaries sometimes causing fish kills (CASQA, 2003). 2. During the construction phase, there is potential for an increase in trash and debris loads due to lack of proper contractor good housekeeping practices at the construction site.	1. Basin Plan narrative floating material objective: "Waters shall not contain floating materials, including solids, liquids, foams, and scum, in concentrations that cause a nuisance or adversely affect beneficial uses." 2. Basin Plan narrative settleable materials objective: "Waters shall not contain suspended or settleable material in concentrations that cause nuisance or adversely affect beneficial uses." 3. Basin Plan narrative Biochemical Oxygen Demand (BOD5) objective: "Waters shall be free of substances that result in increases in the BOD which adversely affect beneficial uses." 4. Basin Plan objectives for dissolved oxygen (DO): "At a minimum (see specifics below), the mean annual dissolved oxygen concentration of all waters shall be greater than 7 mg/L, and no single determination shall be less than 5.0 mg/L, except when natural conditions cause lesser concentrations. The dissolved oxygen concentration of all surface waters designated as WARM shall not be depressed below 5 mg/L as a result of waste discharges."
MBAS (Methylene blue activated substances)	1. MBAS are related to the presence of detergents in water. Positive results may indicate the presence of wastewater or be associated with urban runoff due to commercial and/or residential vehicle washing or other outdoor washing activities. Surfactants disturb the surface tension which affects insects and can affect gills in aquatic life.	1. Basin Plan objective for MBAS: "Waters shall not have MBAS concentrations greater than 0.5 mg/L in water designated (MUN)."
Cyanide	1. Cyanide has been identified by the Los Angeles County Department of Public Works as a constituent of concern for the Santa Clara River based on monitoring conducted at mass emission Station S29	1. The CTR criteria are the applicable water quality objectives for protection of aquatic life (40 CFR 131.38). The CTR criteria are expressed for acute and chronic (4-day average)

Pollutant of Concern [1]	Rationale for Selection / Exclusion	Significance Criteria
	(LACDPW, 2005). Cyanide is used in electroplating, metallurgy, and gold mining. It is also used to make synthetic fibers, plastics, dyes, pharmaceuticals, and pesticides, including fumigants. In addition, cyanide serves as a chemical intermediate in various production processes. Natural cyanides are produced by certain bacteria, fungi, and algae, and they are present in a number of plants and foods as cyanogenic glycosides. Man-made cyanides typically enter the environment from metal finishing and organic chemical industries. Other sources include iron and steel works, municipal waste burning, cyanide-containing pesticides, road deicers, and vehicle exhaust.	conditions; however, only acute conditions are applicable for stormwater discharges because the duration of stormwater discharge is typically less than 4 days in the Project area. CTR freshwater aquatic life protection acute criteria is 22 µg/L.
Bioaccumulation	1. Some pollutants of concern in stormwater runoff, such as metals or pesticides, have the potential to bioaccumulate in aquatic organisms potentially affecting the health of those organism or other species higher up the food chain.	1. Although bioaccumulation is not a pollutant, it is a condition of concern. The Basin Plan objective for bioaccumulation is: "Toxic pollutants shall not be present at levels that would bioaccumulate in aquatic life to levels which are harmful to aquatic life or human health."
Oxygen, Dissolved & BOD (Biochemical oxygen demand)	1. Adequate DO levels are required to support aquatic life. Depressed levels may lead to anaerobic conditions. 2. BOD can result in decreased dissolved oxygen levels affecting beneficial uses such as habitat designations. 3. DO & BOD are correlated to nutrients and other organic compounds and are subsumed by those categories.	1. Basin Plan objective for dissolved oxygen: "The dissolved oxygen content of all surface waters designated as WARM shall not be depressed below 5 mg/L as a result of waste discharges." 2. Basin Plan objective for BOD: "Waters shall be free of substances that result in increases in the BOD which adversely affect beneficial uses."

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Biostimulatory substances	1. Biostimulatory substances include excess nutrients and other compounds that stimulate aquatic growth resulting in impaired aesthetics and water quality impairments such as lowered dissolved oxygen values. 2. Biostimulatory substances are correlated to nutrients and other organic compounds and are subsumed by those categories.	1. Basin Plan objectives for biostimulatory substances: "Waters shall not contain biostimulatory substances in concentrations that promote aquatic growth to the extent that such growth causes nuisance of adversely affects beneficial uses."
Chemical Pollutants	3. Chemical pollutants in excessive amounts in drinking water are harmful to human health. 4. The chemical pollutants referenced under this water quality objective, such as trace metals and nitrate, are either subsumed by the categories above, or are not found in urban runoff (e.g., fluoride).	2. Basin Plan objectives for chemical Pollutants: "Surface waters shall not contain concentrations of chemical Pollutants in amounts that adversely affect any designated beneficial use."
Temperature	1. Elevated temperatures are typically associated with discharges of process wastewaters or non-contact cooling waters. Increase in temperature can result in lower dissolved oxygen levels impairing habitat and other beneficial uses of receiving waters. Stormwater runoff from the Project site is expected to cool somewhat during treatment in structural BMPs and will be diluted in the receiving water. As the beneficial uses in the receiving waters for the Project include warm freshwater habitat to support warm water ecosystems, any increase in temperature resulting from stormwater runoff from the project is expected to be less than significant.	1. Basin Plan objectives for temperature: "For waters designated WARM, water temperature shall not be altered by more than 5 ºF above the natural temperature. At no time shall these WARM-designated waters be raised above 80 ºF as a result of waste discharges".

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Total Residual Chlorine	1. Municipal pools and private pools in areas served by a municipal sanitary system are required to be discharged into the sanitary system. Chlorine disinfection will not take place on the project site and there will not be any sources of elemental chlorine. Chloride sources (e.g. fertilizers or other compounds with salts) are evaluated separately. Therefore, total residual chlorine will not be present in runoff from the project.	1. Basin Plan objectives for total residual chlorine: "Chlorine residual shall not be present in surface water discharges at concentrations that exceed 0.1 mg/L and shall not persist in receiving waters at any concentration that causes impairment for beneficial uses".
Color, Taste, and Odor	1. Undesirable tastes and odors in water may be a nuisance and may indicate the presence of a pollutant(s). Odor associated with water can result from decomposition of organic matter or the reduction of inorganic compounds, such as sulfate. Other potential sources of odor causing substances, such as industrial processes, will not occur as part of the project. Color in water may arise naturally, such as from minerals, plant matter, or algae, or may be caused by industrial pollutants. 2. The Project will contain no heavy industrial uses. Commercial areas of the project are not expected to be a significant source of pollutants that might impart color or odor to stormwater flows from the Project area. Source controls are expected to reduce the amount of plant material and BMPs will reduce sediment which could contribute to color or odor nuisances. Therefore, color-, taste-, or odor-producing substances are not pollutants of concern for the project.	1. Basin Plan objective for color: "Waters shall be free of coloration that causes nuisance or adversely affects beneficial uses". 2. Basin Plan objectives for taste and odor: "Ground waters shall not contain taste or odor-producing substances in concentration that cause nuisance or adversely affect beneficial uses".

Pollutant of Concern [(1)]	**Rationale for Selection / Exclusion**	**Significance Criteria**
Exotic Vegetation	1. Exotic vegetation typically provides little habitat value and can out compete native vegetation that is more suitable habitat for aquatic and terrestrial organisms. 2. The landscape management plan will not use exotic vegetation, and undesirable invasive vegetation will be eradicated to the extent possible. Therefore, exotic vegetation is not a pollutant of concern for the Project.	1. Basin Plan objective for exotic vegetation: "Exotic vegetation shall not be introduced around stream courses to the extent that such growth causes nuisance or adversely affects designated beneficial uses."
Mineral Quality (TDS, Boron, Sulfate, Sodium Absorption Ratio - SAR)	1. LACDPW stormwater monitoring data arithmetic mean concentrations for TDS, sulfate, and boron for urban land uses are below the water quality objectives for minerals. Calculated SAR values are 0.6 for SF residential and 1.9 for commercial based on LACDPW data. The minerals listed in the Basin Plan, except chloride and nitrogen, are not believed to be pollutants of concern due to the absence of river impairments and /or anticipated runoff concentrations below the Basin Plan objectives	1. Basin Plan objectives for minerals: Reach 5 TDS (mg/L) 1000 Sulfate (mg/L) 400 Boron (mg/L) 1.5 SAR (mg/L) 10
pH	1. Mean runoff concentrations in the Los Angeles County stormwater monitoring data ranged from 6.5 for mixed- and single-family residential land uses to 7.0 for commercial land use. Therefore, pH in the Santa Clara River is not expected to be affected by runoff discharges from the project.	1. Basin Plan objective for pH: "the pH of inland waters shall not be depressed below 6.5 or raised above 8.5 as a result of waste discharges. Ambient pH levels shall not be changed more than 0.5 units from natural conditions as a result of waste discharge."

Pollutant of Concern [(1)]	Rationale for Selection / Exclusion	Significance Criteria
PCBs	1. PCBs are highly toxic persistent chemicals that have been historically released into the environment from industrial uses, such as transformers. Due to their persistence, PCBs can still be detected in urban runoff due to historic industrial sources of these chemicals. 2. The project area did not historically include PCB-producing land uses and industrial land uses are not included in the proposed project. Therefore, PCBs are not a pollutant of concern for the project.	1. Basin Plan narrative regarding PCBs: "The purposeful discharge of PCBs to waters of the Region, or at locations where the waste can subsequently reach waters of the Region, is prohibited. Pass-through or uncontrollable discharges to waters of the Region, or at locations where the waste can subsequently reach waters of the Region, are limited to 70 pg/L (30 day average) for protection of human health and 14 ng/L and 30 ng/L (daily average) to protect aquatic life in inland fresh waters and estuarine waters respectively".
Radioactive Substances	1. Some activities such as mining or industrial activities can increase the amount of radioactive substances impairing beneficial uses. 2. The project will not have industrial or other activities that would be a source of any radioactive substances, and development will stabilize any naturally radioactive soils, though unlikely to be present in the project area. Therefore, radioactive substances are not a pollutant of concern for the project.	1. Basin Plan narrative objective for radioactive substances: "Radionuclides shall not be present in concentrations that are deleterious to human, plant, animal, or aquatic life or that result in the accumulation of radionuclides in the food web to an extent that presents a hazard to human, plant, animal, or aquatic life".

1. The pollutants of concern for the water quality analysis are those that are anticipated or potentially could be generated by the project at concentrations, based on water quality data collected in Los Angeles County from land uses that are the same as those included in the Project, that current loadings or historic deposits of the pollutant are impacting the beneficial uses of a receiving water, elevated levels of the pollutant are found in sediments of a receiving water and/or have the potential to bioaccumulate in organisms therein, or the detectable inputs of the pollutant are at concentrations or loads considered potentially toxic to humans and/or flora and fauna.

A.2. References

California Association of Stormwater Quality Agencies (CASQA), 2003. *Stormwater Best Management Practices Handbook New Development & Redevelopment*.

Los Angeles Regional Water Quality Control Board, 1995. Water Quality Control Plan Los Angeles Region Basin Plan for the Coastal Watersheds of Los Angeles and Ventura Counties.

Los Angeles Regional Water Quality Control Board, October, 25th, 2001. Resolution 01-018: Amendment to the Water Quality Control Plan for the Los Angeles Region to Update the Bacteria Objectives for Water Bodies Designated for Water Contact Recreation

Los Angeles Regional Water Quality Control Board, July, 10th, 2003. Resolution R03-008 Revision of interim waste load allocations for chloride in the Amendment to the Water Quality Control Plan for the Los Angeles Region to include a TMDL for Chloride in the Upper Santa Clara River.

Los Angeles Regional Water Quality Control Board, August, 7th, 2003. Resolution R03-011 Amendment to the Water Quality Control Plan for the Los Angeles Region to include a TMDL for Nitrogen Compounds in the Santa Clara River.

US Environmental Protection Agency California Toxics Rule (CTR), 40 C.F.R. §131.38.

APPENDIX B
WATER QUALITY MODEL METHODOLOGY

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B. WATER QUALITY MODEL METHODOLOGY

B.1. Model Description

B.1.1. Model Overview

The model used to assess stormwater quality impacts associated with the proposed Newhall Ranch Landmark Village sub-division is an empirical, volume-based pollutant loads model. This type of loadings model is generally applicable in the planning and evaluation stages of a project. The model was developed to assess the potential impact of development on water quality and to evaluate the effectiveness of the structural Best Management Practices (BMPs) that will treat storm water runoff as part of the project storm water treatment system. Two project conditions were evaluated with the water quality model:

1. Pre-development
2. Post-development with treatment BMPs

Measured runoff volumes and water quality characteristics of storm water are highly variable. To account for this variability, a statistical modeling approach was used to estimate the volume of storm water, the concentration of pollutants in storm water, and the overall pollutant load (total mass of pollutants) in storm water runoff. A statistical description of storm water provides an indication of the average characteristics and variability of the water quality parameters of storm water. It does not forecast runoff characteristics for specific storms or monitoring periods.

The statistical model is based on relatively simple rainfall/runoff relationships and estimated concentrations in storm water runoff. The volume of storm water runoff is estimated using a modification to the Rational Formula, an empirical expression that relates runoff volume to the rainfall depth and the basin characteristics such as imperviousness, and soils infiltration characteristics. The pollutant concentration in storm water runoff is represented by an expected average pollutant concentration, called the event mean concentrations (EMC). The EMCs are estimated from available monitoring data from, and are strongly dependent on the land-use type.

The flow chart in Figure B-1 provides an overview of the modeling methodology.

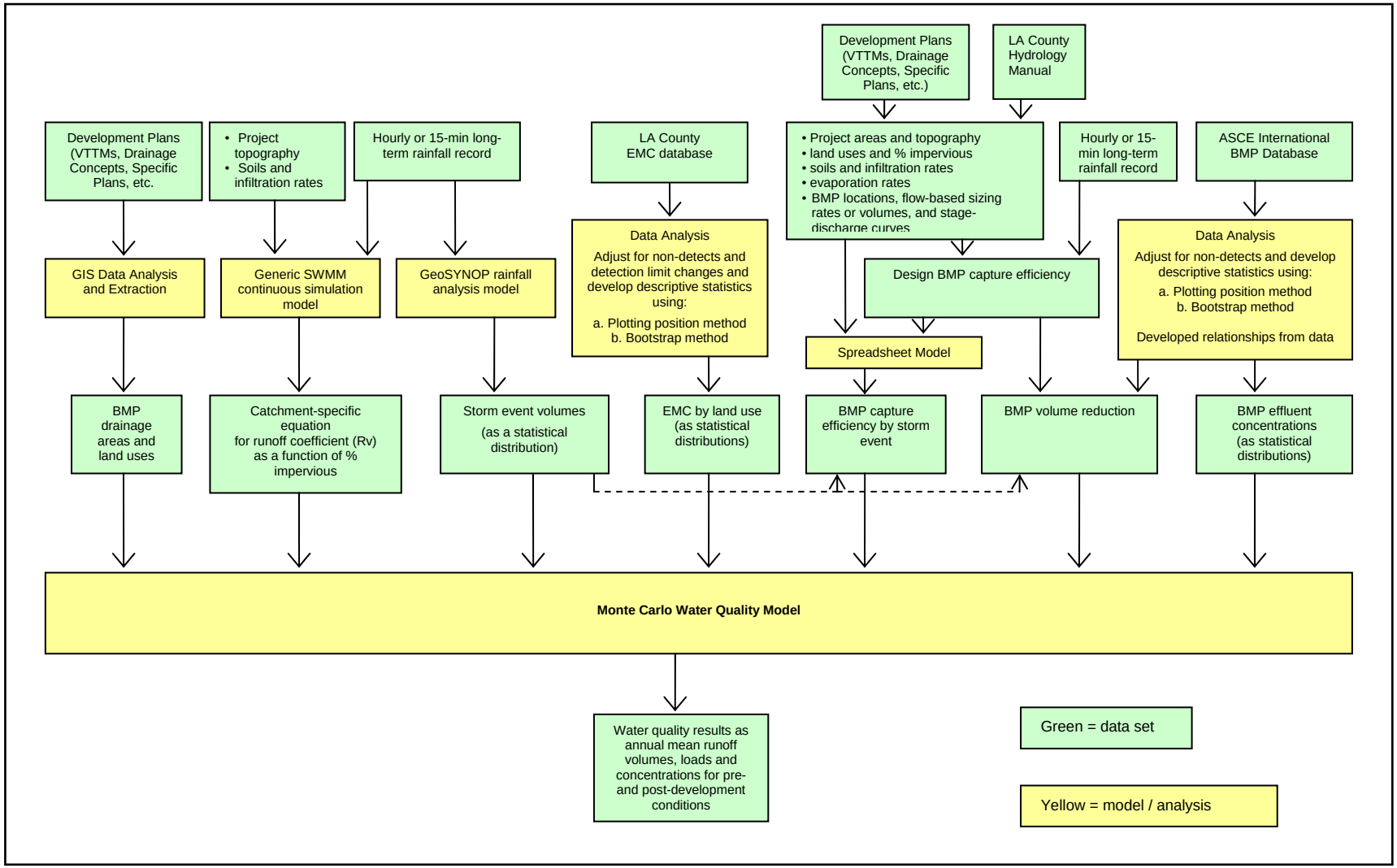

Figure B-1: Overview of Water Quality Analysis Methodology

The model does not incorporate the hydraulics or hydrology of the site, which would be more appropriate for design stages and requires additional data and more sophisticated modeling. The model includes water quality benefits achieved by structural BMPs but not source control BMPs because data is generally not available or conclusive for the latter. Model results are presented for average annual runoff volumes, pollutant loads, and pollutant concentrations.

As with all environmental modeling, the precision of results is heavily dependent on how well the hydrologic, water quality and BMP effectiveness data describe the actual site characteristics. Local and regional data are used to the fullest extent possible to help minimize errors in predictions, but such data are limited and traditional calibration and verification of the model is not feasible. It is important to note that the predictions of relative differences should be more accurate than absolute values.

B.1.2. Model Assumptions

The water quality modeling methodology requires that some assumptions are made for both the model input parameters and the way the modeling calculations are carried out. Section B.2.6 discusses the assumptions that were made in specifying the model parameters and Section B.3.4 discusses the assumptions regarding the modeling approach. Section B.4 discusses model accuracy.

B.2. Model Input Parameters

Many parameters that can affect pollutant loads and concentrations vary spatially and may not be adequately represented by stormwater monitoring data collected at discrete locations. Examples include source concentrations, topography, soil type, and rainfall characteristics all of which can influence the buildup and mobilization of pollutants. The following model parameters represent the best data currently available for representation of existing and developed site conditions in the water quality model.

B.2.1. Storm Events

Rainfall analysis was conducted with data from the National Climatic Data Center (NCDC) Newhall rain gauge (station number 046162), located in the town of Newhall, California. Figure B-2 shows the location of the Newhall gauge in relation to the Newhall Ranch Landmark Village Project area. This gauge is located approximately 7 miles from the project. The gauge elevation of 1,243 ft above mean sea level (AMSL) is comparable to the Landmark Village Project area elevation of approximately 1,000-1,200 ft AMSL.

While the period of record rainfall data collected at the Newhall rain gauge is quite long (35 years), there are still some gaps in the record. In order to improve the characterization of rainfall at the project site, estimates of the missing rainfall data were made through correlation of the Newhall rain gauge with the San Fernando rain gauge (NCDC station number 047762) which is located approximately 5 miles away (south and slightly east).

The Castaic Junction gauge monitored by the Los Angeles County Department of Public Works (LACDPW) is located closer to the Project; however, the usable period of record at this gauge is limited to approximately 12 years which is considered too short to produce significant results in long-term simulation.

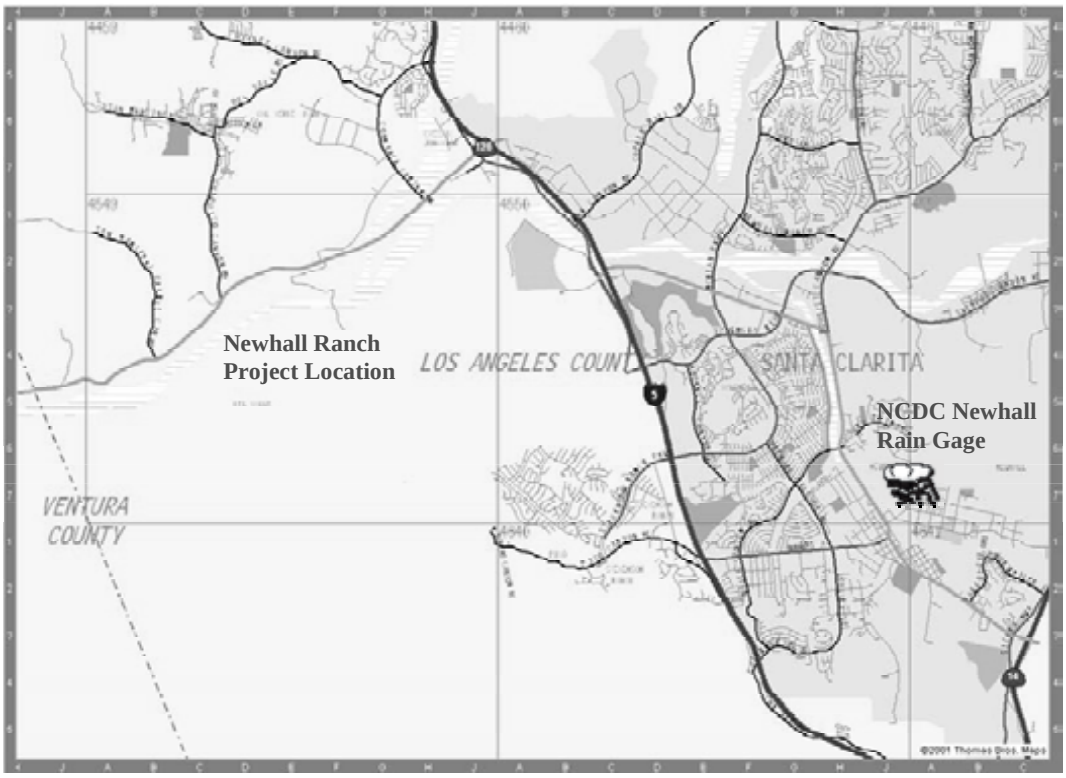

Figure B-2: Location of Newhall Rain Gauge in the Vicinity of the Project Area

First a comparison of daily rainfall totals was made from the available data to assess the similarity in rainfall amounts between the two stations. Daily data from 1969 to 2003 was screened to keep only the 24-hour totals with measured rainfall at both stations, which eliminated missing data at either station. Correlation of the 24-hour rainfall totals is shown in Figure B-3.

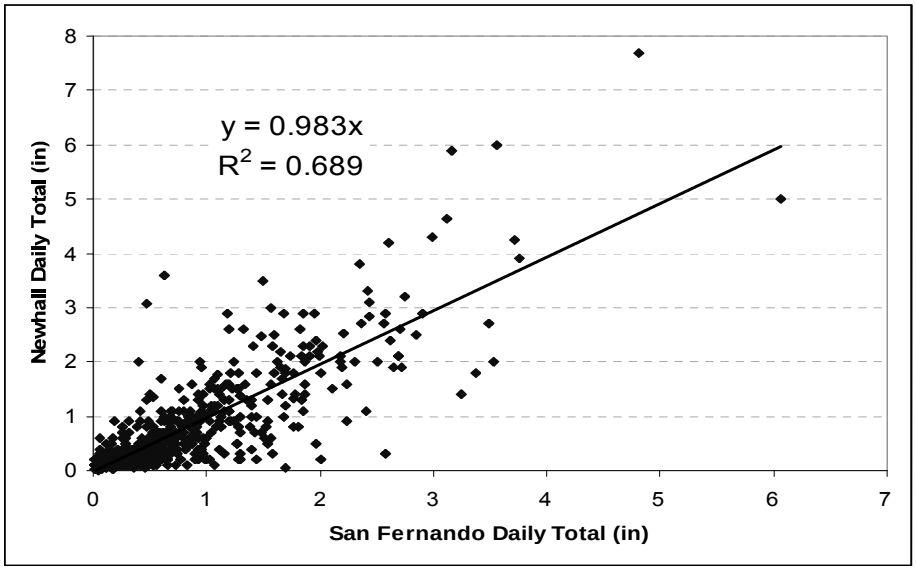

Figure B-3: Correlation of 24-hour Totals between Newhall & San Fernando Gauges

The correlation is reasonably strong considering that the comparison is between the daily accumulations, i.e. a storm could result in appreciable rainfall at one gauge and little rainfall at the other. This comparison indicates that daily precipitation depths are similar between the two gauges. Another comparison was made using only months with a complete rainfall record and measured rainfall at both stations (Figure B-4). This monthly correlation was much stronger due to the longer comparison period, and indicated slightly higher rainfall amounts at the Newhall gauge compared to the San Fernando gauge.

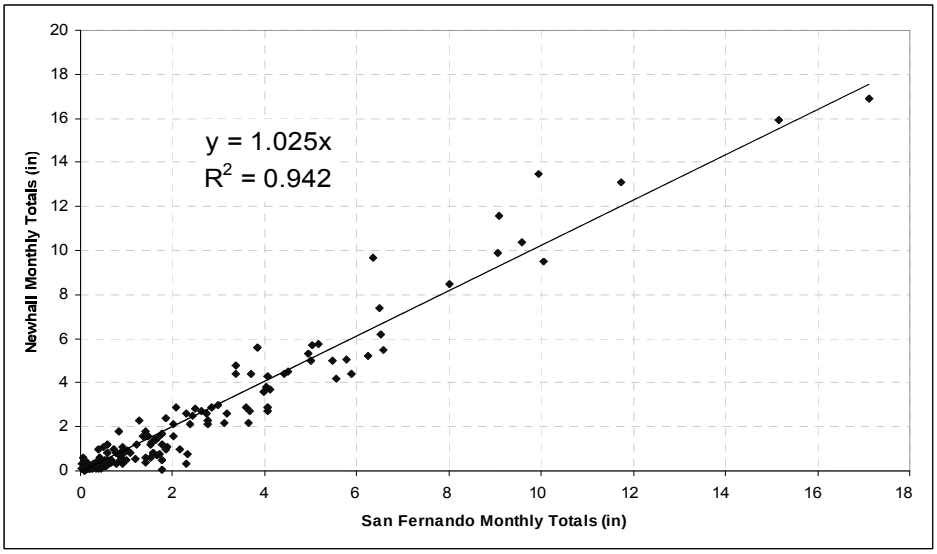

Figure B-4: Correlation of Monthly Totals Newhall & San Fernando Gauges

Based on the relationship developed through the monthly comparison, a multiplier of 1.025 was applied to the hourly rainfall data from the San Fernando gauge to fill in the missing periods of

rainfall data at the Newhall gauge. Values were rounded to the nearest 1/100 inch after the adjustment.

Rainfall analysis was conducted for all storm events and for the storms that are expected to contribute to stormwater runoff (storms > 0.1 inches). The rainfall data were analyzed using a code similar in performance to EPA's Synoptic Rainfall Analysis Program (SYNOP). The customized code (GeoSYNOP) was used as it facilitates filling in missing periods of data and is more robust when handling the date and time of storms. GeoSYNOP subdivides the rainfall record into discrete events separated by a dry inter-event period, which in this case was set to a minimum of 6 hours. Small rainfall events whose depth was less than or equal to 0.10 inches were deleted from the record as such events tend to produce little if any runoff (USEPA, 1989; Schueler, 1987). For the Newhall gauge, a total of 538 storm events (> 0.1 inches) were segregated from the continuous data. Storm statistics for the full (all the storms) and the trimmed (storms > 0.1 inch) data sets are shown in Table B-1.

Table B-1: Analysis Results for the Actual and Filled Newhall Rainfall Data

Storms	Newhall Gauge 1969 – 2003	Original Record	Augmented Record[1]
	Total Missing Records (days):	427	52
All Storms	Average annual rainfall (in):	17.4	18.8
	Total number of storms:	840	890
	Average number of storms per year:	24.0	25.4
	Average storm volume (in):	0.72	0.74
	Average storm duration (hrs):	6.87	7.35
	Average storm intensity (in/hr):	0.103	0.101
Storms >0.1 inch	Average annual rainfall (in):	16.2	17.9
	Total number of storms:	493	538
	Average number of storms per year:	14.1	15.4
	Average storm volume (in):	1.15	1.16
	Average storm duration (hrs):	11.0	11.5
	Average storm intensity (in/hr):	0.107	0.105

1- Augmented record includes adjusted data from San Fernando gauge to fill gaps in Newhall gauge record.

B.2.2. Runoff Coefficients

One of the most variable parameters is the runoff coefficient, which is a function of the percent impervious. Novotny and Olem (1994), when discussing the Rational Formula, state "...the runoff coefficient is the most important task of the entire calculation." The following describes how the runoff coefficients were estimated in the model.

B.2.2.1. SWMM Runoff Coefficient Modeling Parameters

The Water Quality model uses a linear equation to estimate a runoff coefficient for sub-basins as a function of the percent impervious. The format of this equation is described as:

Runoff Coefficient = Slope × % Impervious + Intercept

The appropriate slope and intercept to define the runoff coefficient equation may be taken from region-specific data, regulatory guidance or developed using hydrologic models. The Los Angeles County Standard Urban Stormwater Mitigation Plan (SUSMP) Manual and the LA County Hydrology Manual use the following equation to calculate developed runoff coefficient:

$$C_D = (0.9 \times \text{Imperviousness}) + (1.0 - \text{Imperviousness}) \times C_U$$

Where: C_D = Developed Runoff Coefficient

Imperviousness = Proportion Impervious (0 to 1)

C_U = Undeveloped Runoff Coefficient

The undeveloped runoff coefficient (C_U) in this equation is a function of soil type and rainfall intensity. For most soils found in LA County area and the range of intensities associated with water quality storms, C_U may be assumed to equal 0.1. Substituting this value into the equation above yields:

Runoff Coefficient = 0.008 × % Impervious + 0.1

Note: This equation was <u>not</u> used in water quality modeling. It was only used as a basis for comparison with project-specific runoff coefficient equations developed as described below.

As the Landmark Village Project area contains a variety of soil conditions, continuous simulation modeling, using the Storm Water Management Model (SWMM), was conducted to determine the appropriate slope and intercept parameters to use in the linear runoff coefficient equation. Key parameters for the SWMM model are shown in Table B-2.

Table B-2: SWMM Runoff Module Parameters

SWMM Runoff Parameters	Units	Values
Wet time step	seconds	600
Wet/dry time step	seconds	600
Dry time step	seconds	14,400
Impervious Manning's n		0.012
Pervious Manning's n		0.25
Drainage area modeled for Rv determination	acres	10 (Actual project areas sub-divided by hydrologic soil groups and normalized to 10 acre catchments)
Shape		Rectangular, 500 ft flow path length for pervious areas, 250 ft flow path length for impervious area.
Impervious Fractions Modeled		0%, 33.3%, and 100%. See Table B-3 for specific runoff block dimensions.
Slope	ft/ft	0.02, project area is relatively flat.
Evaporation	in/month	80% of reference ET values contained in

SWMM Runoff Parameters	Units	Values
		Table B-5 were used for the existing site conditions to reflect existing uses and the post-development project condition.
Soil properties / infiltration		Green-Ampt soil parameters as contained in Table B-4.
Depression storage, impervious	inches	0.02, based on Table 5-14 in SWMM manual (James and James, 2000).
Depression storage, pervious	inches	0.06, based on Table 5-14 in SWMM manual (James and James, 2000).

Runoff path lengths will affect ET and runoff volumes. As the path length increases, ET and infiltration increase and runoff decreases. For consistency in model runs three scenarios were modeled as shown in Table B-3 with consistent runoff path lengths for pervious surfaces and impervious surfaces. Rectangular catchments were assumed, thus the catchment width for input to SWMM was calculated as the catchment area divided by the total path length. As only one width may be assigned for each catchment, modeled impervious fractions were chosen specifically to result in consistent runoff path lengths for pervious and impervious surfaces. Maintaining consistent path lengths ensures that the results of SWMM can be well-approximated by a linear trendline.

Table B-3: SWMM Runoff Block Modeled Percent Impervious Values

Area (ac)	% Impervious	SWMM Width (ft)	Pervious Flow Length (ft)	Pervious Flow Length (ft)
10	0	871	500	0
10	33.3	581	500	250
10	100	1742	0	250

Some soils in the vicinity of the project area have been classified as sandy-loam soils deposited by the Santa Clara River (R.T. Frankian & Associates, 2001). Soils in the Landmark Village Project area were conservatively modeled with infiltrative capacity comparable to silt-loam soils, resulting in little surface runoff for the existing condition and a conservative estimate for the developed condition when further reducing the hydraulic conductivity by 25 percent to account for compaction. The Green-Ampt soils properties used for the SWMM modeling are shown in Table B-4.

Table B-4: Green-Ampt Soil Parameters

Soil Texture Class	Suction Head (cm)	Ks (cm/hr)	Suction Head (in)	Ks (in/hr)
Silt Loam – Existing Condition	16.68	0.68	6.57	0.27
Silt Loam – Developed Condition	16.68	0.51	6.57	0.20

Soil properties estimated from information contained in Table 5.5.5 of the Handbook of Hydrology (Maidment, ed. 2003)

Reference ET values for estimating actual ET rates was taken from Figure B-5 produced by the California Department of Water Resources. The Landmark Village Project site is located in zone 14. Reference ET values for zone 14 are reproduced in Table B-5.

Existing site conditions consist of agricultural row crops, both irrigated and dry farming. To represent average existing site conditions, 80% of the reference ET values were used to reflect dry farming crops with lower water requirements and irrigated farming with slightly higher evapotranspiration rates. Eighty percent of the reference ET values were also used to simulate the landscaped areas in the post-development condition.

Table B-5: Evaporation Parameters for Hydrology Model (from CA ET map)

Month	Evapotranspiration Rates			80%
	Inch / day	Days / month	Inch / Month	Inch / Month
January	0.05	1.24	1.55	1.24
February	0.08	1.79	2.24	1.79
March	0.12	2.98	3.72	2.98
April	0.17	4.08	5.1	4.08
May	0.22	5.46	6.82	5.46
June	0.26	6.24	7.8	6.24
July	0.28	6.94	8.68	6.94
August	0.25	6.2	7.75	6.2
September	0.19	4.56	5.7	4.56
October	0.13	3.22	4.03	3.22
November	0.07	1.68	2.1	1.68
December	0.05	1.24	1.55	1.24
	Total	45.63	57.04	45.63

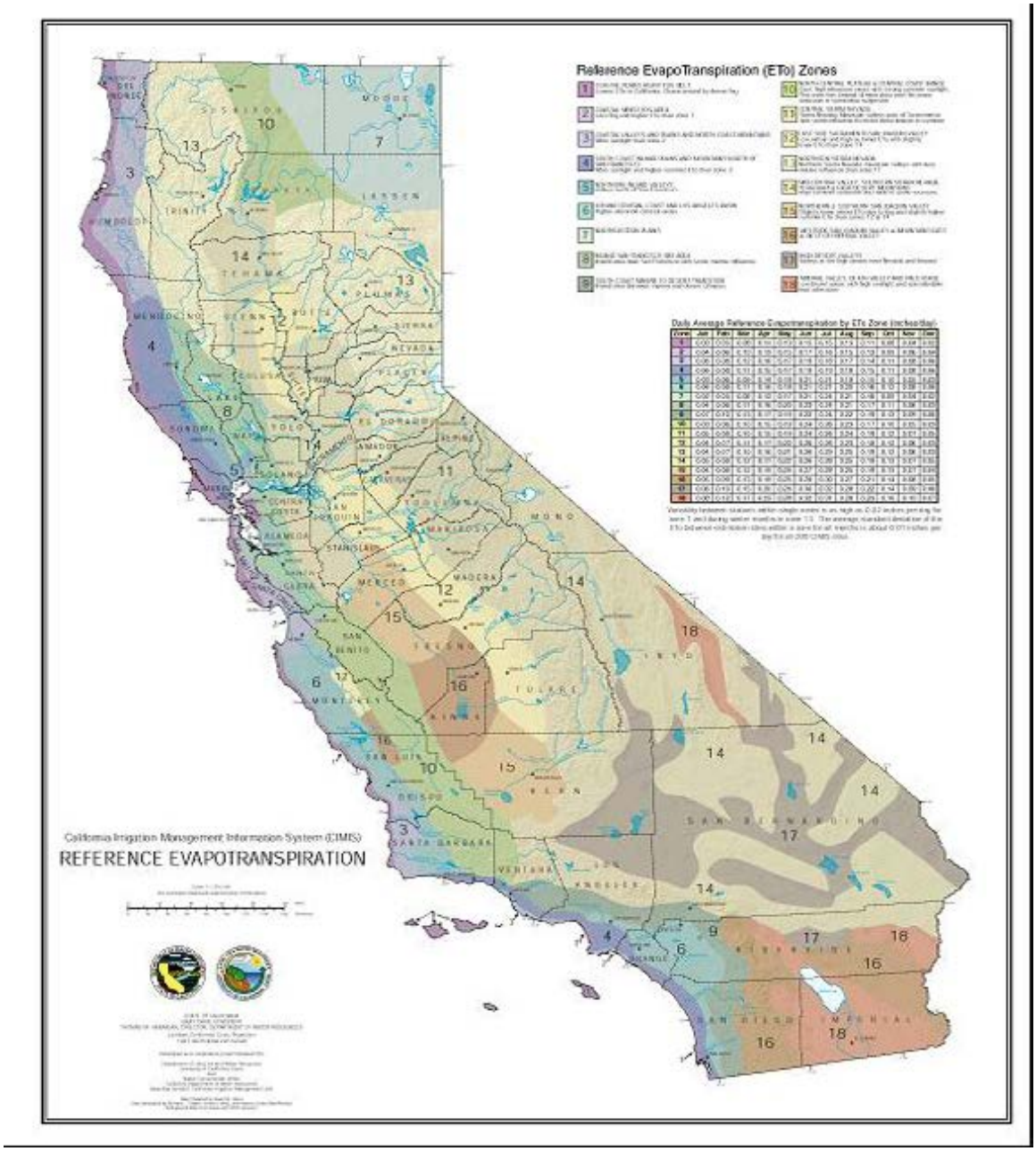

Figure B-5: Reference ET for CA Zones

B.2.2.2. SWMM Runoff Coefficient Results

Using the model input parameters described above, runoff coefficient equations have been developed for all modeled watersheds. Figure B-6 and Figure B-7 display the SWMM results (as diamonds) and the best fit line for existing and developed project conditions, respectively.

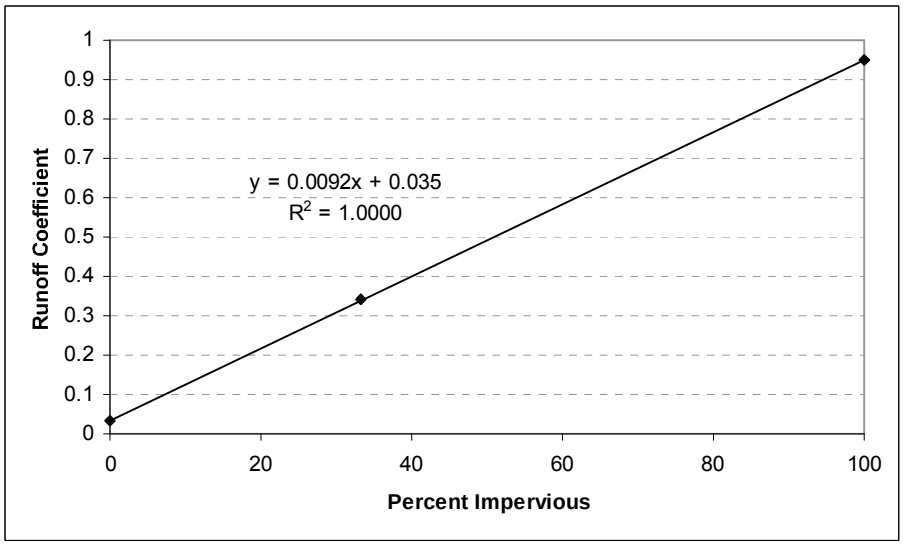

Figure B-6: Existing Conditions Runoff Coefficient Equation

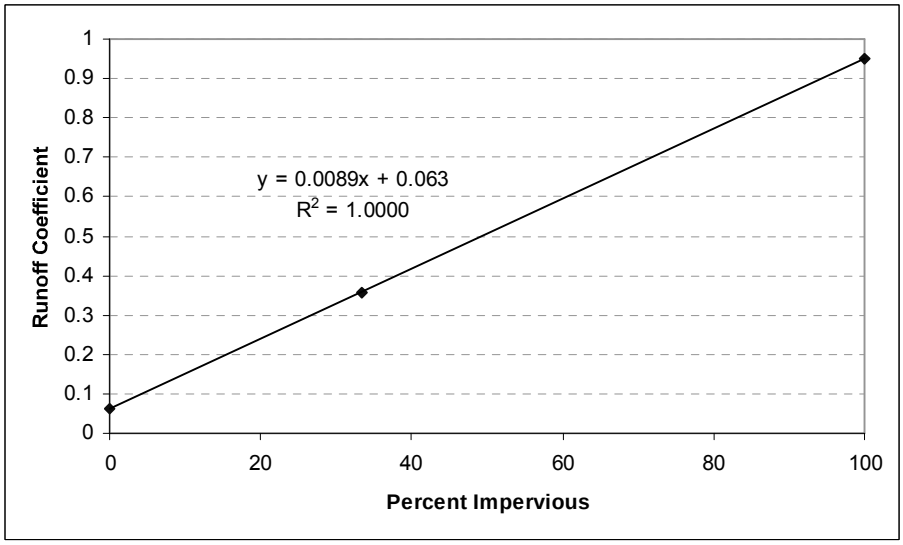

Figure B-7: Developed Conditions Runoff Coefficient Equation

The intercept was rounded to three decimal places resulting in the following equations used to estimate runoff coefficients in the water quality model as a function of imperviousness

- Existing Conditions: Runoff Coefficient = 0.0092 × % Impervious + 0.035
- Developed Conditions: Runoff Coefficient = 0.0089 × % Impervious + 0.063

B.2.3. Land Use & Treatment BMPs

The delineation of land uses and areas within Landmark Village were determined from the Vesting Tentative Tract Map No. 53108 for the developed Project conditions. The existing and developed conditions of the Project and associated off-site areas are summarized in Table B-6 and Table B-7, respectively.

Included in the water quality analysis are 103.6 acres of off-site project areas. The section of State Route 126 (SR126) adjacent to the Landmark Village Project area will undergo widening, which will increase the area of this section of SR126 from 72.8 to 95.6 acres. Four water tanks and access roads will be located near the project. The required area for the water tanks was conservatively estimated at 2 acres per tank and access road, as detailed information is not yet available. Modeled areas for SR126 for existing and developed conditions are shown in Table B-6 and Table B-7, respectively.

Table B-6: Existing Conditions Project & Off-site Areas

Land Use	Development Area (Acres)		
	Project Site	Off-Site	
		SR126	Water Tanks
Row Crops	292.6		
Roads		72.8	
Open Space		22.8	8
Total	292.6	95.6	8

Table B-7: Developed Conditions Project & Off-site Areas

Land Use	Development Area (Acres)		
	Project Site	Off-Site	
		SR126	Water Tanks
Single family	50		
Multi-family	60.7		
Apartments	21		
Commercial	36.5		8
Elementary School	9		
Parks	16.1		
Recreation Centers	5.2		
Trails & Misc.	38.3		
Roads	55.8	95.6	
Total	292.6	95.6	8

Table B-8 provides the modeled land uses and percent impervious values used to represent the existing and developed project and off-site conditions. The modeled land uses were based on the most representative land use within the available data sets (see Section B.2.4). If a representative land use was not available from the monitored land uses (Section B.2.4), then a conservative land use was used. For example, the water tanks and associated access roads will have very low

traffic and are not expected to result in appreciable pollutant loads in stormwater runoff. The commercial land use, while expected to be very conservative for representing the water tanks and access roads, is believed to most closely simulate these types of land uses and was therefore selected to represent runoff from these areas.

Table B-8: Modeled Land Uses, Percent Imperviousness, & Data Source

Land Use	Modeled Percent Impervious	Modeled Land Use
Row Crops	15%	Agriculture
Single family	42%	Single Family Residential
Multi-family	68%	Multi-family Residential
Apartments	68%	Multi-family Residential
Commercial	92%	Commercial
Elementary School	82%	Education
Parks	15%	Open Space
Recreation Centers	15%	Open Space
Trails & Misc.	5%	Open Space
Roads	100%	Transportation

[1] Percent impervious values are based on the LA County Hydrology Manual.

Stormwater runoff from all developed areas within the Landmark Village Project will be routed to bioretention areas, vegetated swales, and/or extended detention basin treatment control BMPs. Catch basin inserts will also be used in high use parking lots. The off-site SR-126 expansion Project will provide vegetated swale treatment for both the new and existing untreated roadway area. The utility corridor maintenance access road and potential future trail, as well as the water tanks and access roads, will drain to biofiltration treatment (vegetated swale or filter strip) or bioretention treatment. The extended detention basin, vegetated swales, and bioretention areas will be designed to operate off-line, receiving dry weather flows, small storm flows, and the initial portion of large storm flows from a low-flow diversion structure in the storm drain. The effectiveness of treatment BMPs is evaluated without taking site design and source control BMPs into account. Therefore, the analysis is conservative in that it understates water quality controls.

B.2.4. Stormwater Runoff Pollutant Concentrations

Stormwater monitoring data collected by LACDPW was used to derive estimates of pollutant concentrations in runoff from urban land uses. The existing conditions of the Landmark Village Project site contain agricultural uses. Stormwater monitoring data collected by Ventura County was used to estimate stormwater pollutant concentrations for agricultural land use.

B.2.4.1. Los Angeles County Monitoring Data

Recent and regional land-use based stormwater quality monitoring data was collected through the LA County Stormwater Monitoring Program. This program was initiated with the goal of providing technical data and information to support effective watershed stormwater quality management programs in Los Angeles County. Specific objectives of this project included monitoring and assessing pollutant concentrations from specific land uses and watershed areas. In order to achieve this objective, the County undertook an extensive stormwater sampling project that included 8 land use stations and 5 mass emission stations (located at the mouths of major streams and rivers), which were tested for 82 water quality constituents. These data are presented in *Los Angeles County 1994-2000 Integrated Receiving Water Impacts Report, 2000* and *Los Angeles County 2000-2001 Stormwater Monitoring Report, 2001*.

Stormwater quality for the Newhall Ranch and the Landmark Village sub-division was estimated based on the recent EMC data collected by LA County (LA County, 2000). These data were used because of the relatively close location to the project site and because the monitored land uses were representative of the proposed land uses for the Newhall Ranch Project. The monitored land uses stations are listed in Table B-9 with a brief description of the site and when the monitoring data were collected.

Table B-9: LA County Land Use Monitoring Stations Available for Water Quality Modeling

Station Name	#	Modeled Land Use	Site Description[1]	Years Monitoring Conducted
Santa Monica Pier	S08	Commercial	The monitoring site is located near intersection of Appian Way and Moss Avenue in Santa Monica. The storm drain discharges below the Santa Monica Pier. Drainage area is approximately 81 acres. The Santa Monica Mall and Third St. Promenade dominate the watershed with remaining land uses consisting of office buildings, small shops, restaurants, hotels and high-density apartments.	1995-1999
Sawpit Creek	S11	Open Space (& Parks)	Located in Los Angeles River watershed in City of Monrovia. The monitoring station is Sawpit Creek, downstream of Monrovia Creek. Sawpit Creek is a natural watercourse at this location. Drainage area is approximately 3300 acres.	1995-2001
Project 620	S18	Single Family Residential	Located in the Los Angeles River watershed in the City of Glendale. The monitoring station is at the intersection of Glenwood Road and Cleveland Avenue. Land use is predominantly high-density, single-family residential. Drainage area is approximately 120 acres.	1995-2001
Project 1202	S24	Light Industrial	Located in the Dominguez Channel/Los Angeles Harbor Watershed in the City of	1995-2001

Station Name	#	Modeled Land Use	Site Description[1]	Years Monitoring Conducted
			Carson. The monitoring station is near the intersection of Wilmington Avenue and 220th Street. The overall watershed land use is predominantly industrial.	
Dominguez Channel	S23	Freeway (Roadways)	Located within the Dominguez Channel/Los Angeles Harbor watershed in Lennox, near LAX. The monitoring station is near the intersection of 116th Street and Isis Avenue. Land use is predominantly transportation and includes areas of LAX and Interstate 105.	1995-2001
Project 474	S25	Education (Schools)	Located in Los Angeles River watershed in the Northridge section of the City of Los Angeles. The monitoring station is located along Lindley Avenue, one block south of Nordoff Street. The station monitors runoff from the California State University of Northridge. Drainage area is approximately 262 acres.	1997-2001
Project 404	S26	Multi-Family Residential	Located in Los Angeles River watershed in City of Arcadia. The monitoring station is located along Duarte Road, between Holly Ave and La Cadena Ave. Drainage area is approximately 214 acres.	1997-2001

1) Los Angeles County 1999-2000 Draft Stormwater Monitoring Report (Los Angeles County, 2000)

B.2.4.2. Ventura County Monitoring Data

As part of its NPDES permit, the Ventura County Flood Control District conducts monitoring to determine the water quality of stormwater runoff from areas with specific land uses. One monitoring station, Wood Road at Revolon Slough (site A-1), drains the approximately 350 acre Oxnard Agricultural Plain, which is comprised almost entirely of agricultural land (primarily row crops), including a small number of farm residences and ancillary farm facilities for equipment maintenance and storage. Data from the Wood Road station was used to estimate pollutant concentrations in stormwater runoff for agricultural land use.

Land use runoff sampling for the Ventura County stormwater monitoring program originally began during the 1992/93 monitoring season, with up to several samples collected at each site during each storm season. For the A-1 site, the period of record begins during the 1996/97 storm season, and continues through the 2003/04 season. All land use monitoring sites are equipped with automated monitoring equipment, including flowmeters (with area-velocity probes and level sensors) and refrigerated auto-samplers which enable the collection of flow-weighted composite samples. Stormwater quality monitoring data for the agricultural land use site was provided by Mark Davis of the Ventura County Watershed Protection District. This information was extracted from their newly-constructed water quality database, which contains monitoring data for their land use, mass emission, and receiving water monitoring sites.

B.2.4.3. Data Analysis for Derivation of Land Use EMCs

The County of Los Angeles Department of Public Works (LACDPW) has monitored stormwater runoff quality from various land uses throughout the County on an annual basis beginning in 1995 through 2001. For each year of monitoring several storm event mean concentrations (EMCs) are reported and included in the County's annual water quality report to the Los Angeles Regional Water Quality Control Board. The convention for dealing with the censored data (e.g., data only known to be below the analytical detection limit) is to substitute ½ of the detection limit for all non-detects. L.A. County has followed this convention when providing summary arithmetic statistics of the stormwater monitoring data. This method tends to introduce bias into the estimate of the mean and standard deviation and the summary statistics are not believed to be robust or adequately account for non-detects. To further complicate matters, the detection limit for dissolved copper and total lead has changed during the period stormwater monitoring was conducted by LACDPW.

In an effort to provide more reliable and accurate estimates of land use EMCs for the Landmark Village water quality modeling, a robust method of estimating descriptive statistics for censored data with multiple detection limits was employed. The plotting position method described in Helsel and Cohn (1988) was used to estimate censored values using the distribution of uncensored values. Descriptive statistics were then estimated using the parametric bootstrap method suggested by Singh, Singh, and Engelhardt (1997).

Example Data Set

To illustrate the statistical methods used to obtain land use EMCs, the LACDPW stormwater monitoring data collected for total lead from the transportation land use station is used. The data were collected from 01/1996 to 04/2001. At the beginning of March 1997 the detection limit for total lead changed from 10 to 5 μg/L. Table B-10 describes the data according to the number of censored and uncensored values in the example data set.

Table B-10. Number of Censored and Uncensored Data Points in the Total Lead Transportation Land Use Data Set

Total Lead EMC Data for Transportation Land Use	
Uncensored	37
Censored < 10 μg/L	2
Censored < 5 μg/L	38
Total Data Count	77

Prior to applying the plotting position method, it is necessary to check the normality of the data. Figure B-8 shows histograms and probability plots of the transportation land use total lead data above detection limits in normal and lognormal space. As indicated in the figure, the data tends to follow a lognormal distribution, a finding that is common with many pollutants in stormwater.

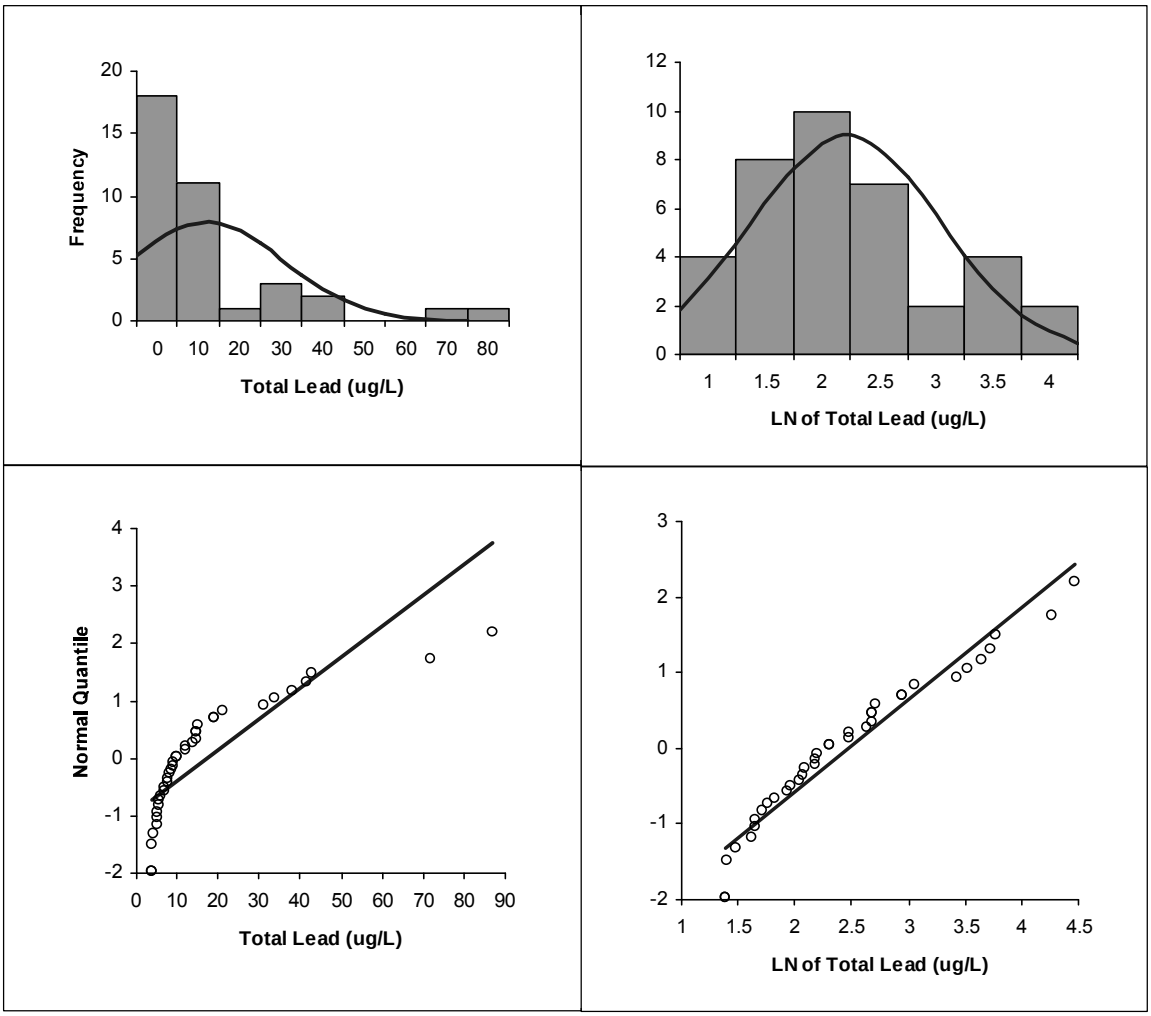

Figure B-8: Histograms and Probability Plots of Transportation Total Lead Data in Arithmetic and Lognormal Space

To verify the visual check that the data are lognormally distributed, the Shapiro-Wilk goodness-of-fit test was used (Royston, 1992). In this test, if $p > 0.1$, the null hypothesis that the log data follow a normal distribution cannot be rejected. For this example data set, the p-value of the log-transformed uncensored data is 0.293, which indicates that lognormal distribution is a good approximation of the distribution of the data set.

Method for Dealing with Multiple Detection Limits

To account for the multiple detection limits in the censored data sets, a regression on order statistics (ROS) method was employed. ROS is a category of robust methods for estimating descriptive statistics of censored data sets that utilize the normal scores for the order statistics (Shumway et al. 2002). The plotting position method by Hirsch and Stendinger (1987) (summarized by Helsel and Cohn, 1988) was the ROS method used. In this method, plotting positions are based on conditional probabilities and ranks, where the ranks of the censored (below detection) and uncensored data (above detection) related to each detection limit are ranked independently. The method is summarized in the equations below.

After plotting positions for the censored and uncensored values have been calculated, the uncensored values are plotted against the z-statistic corresponding to the plotting position and the best-fit line of the known data points is derived. Using this line and the plotting positions for the uncensored data, the values for the uncensored data are extrapolated. Figure B-9 illustrates the results of the application of the plotting position method on the total lead data for transportation land use.

$$pe_j = pe_{j+1} + \frac{A_j}{(A_j + B_j)} \times (1 - pe_{j+1}) \quad (1)$$

Where:

A_j = the number of uncensored observations above the j detection limit and below the j +1 detection limit.

B_j = the number of censored and uncensored observations less than or equal to the j detection limit.

pe_j = the probability of exceeding the j threshold for j = m, m -1, … 2, 1 where m is the number of thresholds; by convention $pe_{m+1} = 0$.

Equation 2 was used for plotting the uncensored data and equation 3 was used for plotting the censored data; the plotting positions of the data were calculated using the Weibull plotting position formula.

$$p(i) = (1 - pe_j) + \frac{(pe_j - pe_{j+1}) \times r}{(A_j + 1)} \quad (2)$$

Where:

p(i) = the plotting position of the uncensored i data point.

r = the rank of the ith observation of the A_j observations above the j detection limit.

$$pc(i) = \frac{(1 - pe_j) \times r}{(n_j + 1)} \quad (3)$$

Where:

pc(i) = the plotting position of the censored i data point.

r = the rank of the ith observation of the n_j censored values below the j detection limit.

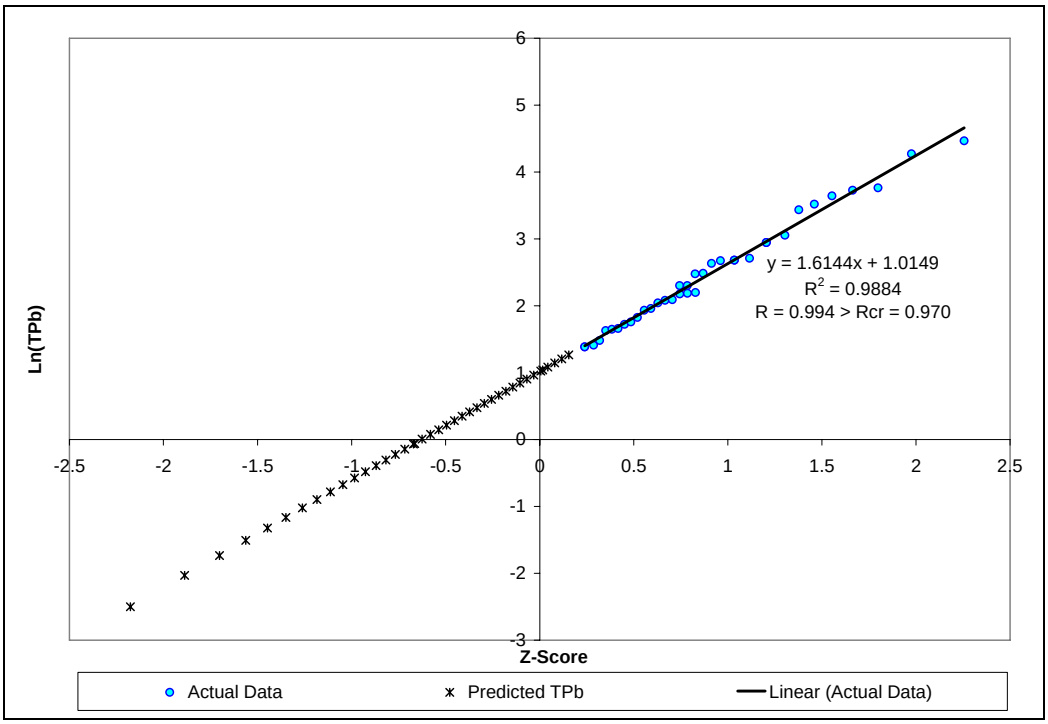

Figure B-9: Probability Plot of the Uncensored and Predicted (Censored) Total Lead Transportation EMCs

Method for Calculating Descriptive Statistics

After the censored data are estimated (or for datasets without non-detects), descriptive statistics were computed using the bootstrap method (Singh et al. 1997). The bootstrap method samples from the data set with replacement several thousand times and calculates the desired descriptive statistics from the sampled data. The steps of the bootstrap estimation method are described below.

1. Take a sample of size *n* with replacement (the sampled data point remains in the data set for subsequent sampling) from the existing data set (Singh et al. recommends n be the same size as the original data set, this recommendation was followed for the analysis) and compute the descriptive statistic, θ_i, from the sampled data.
2. Repeat Step 1 independently N times (10,000 for this analysis) each time calculating a new estimate for θ_i.
3. Calculate the bootstrap estimate θ_B by averaging the θ_i's for i=1 to N

Fundamentally, the bootstrap procedure is based on the Central Limit Theorem (CLT), which suggests that even when the underlying population distribution is non-normal, averaging produces a distribution more closely approximated with normal distribution than the sampled distribution (Devore 1995). Figure B-10 compares the total lead data after estimating censored values using the ROS method described prior to applying the bootstrap method with bootstrapped means of the ROS data. Note the bootstrap means are more normally distributed than the original data and the central tendency of the data is centered near 8 ug/L.

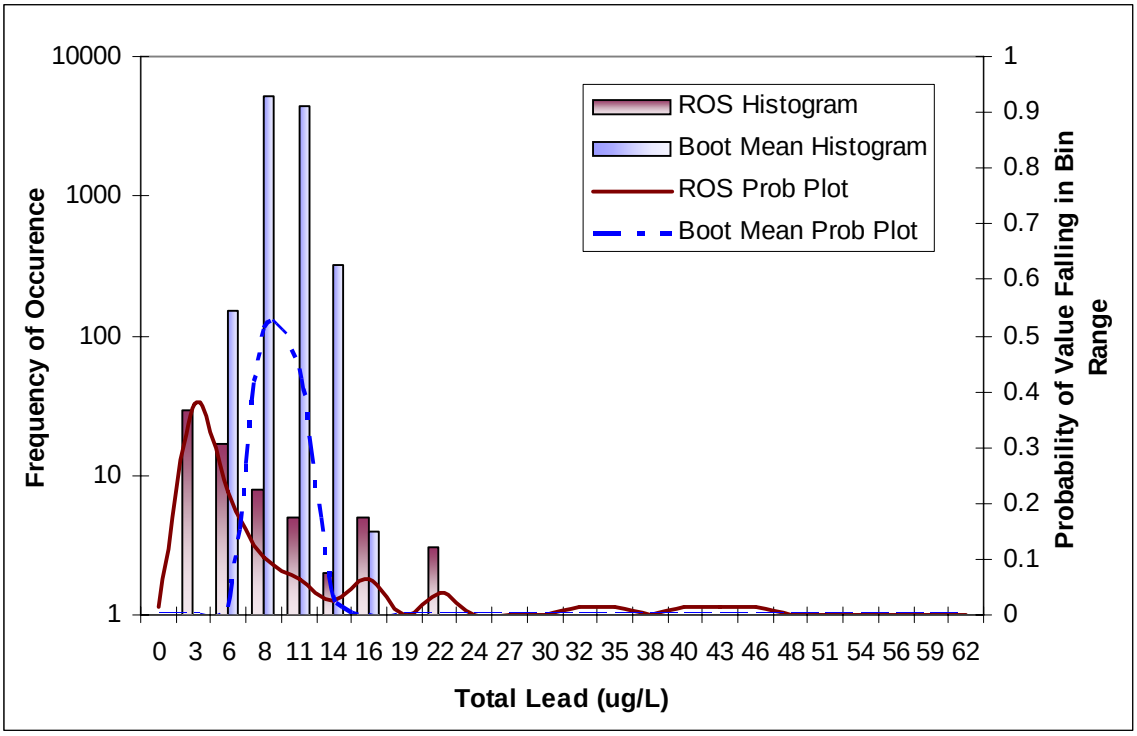

Figure B-10: Comparison of the Distribution of ROS Method Total Lead Data and the Bootstrap Means of the ROS Data.

The majority of the LACDPW stormwater monitoring for the pollutant land use combinations analyzed fit a lognormal distribution. The data that did not statistically fit the lognormal distribution were more closely approximated with a lognormal distribution than a normal distribution. The bootstrap method was applied differently depending on the distributional fit of the data. If the pollutant EMC data for a particular land use fit a lognormal distribution according to the Shapiro-Wilk goodness-of-fit test, the log-transformed data were bootstrapped and an estimate of the mean and standard deviation were obtained in log space and then converted to arithmetic space. The assumption of lognormality was more stringently applied than normal by using an alpha significance value of 0.1. This was done to improve the estimate of the standard deviation when the hypothesis of lognormality is rejected. When analyzing data in log space there is a tendency to overestimate the standard deviation for relatively symmetric data and underestimate the standard deviation for severely skewed data. For datasets that did not fit the lognormal distribution, the raw data were bootstrapped to obtain the mean and standard deviation statistics. Bootstrapping the data in arithmetic space assumes no distribution in those instances when a distribution could not be confirmed through goodness-of-fit testing.

Conclusions

The plotting position method for multiple detection limits has been used in conjunction with the bootstrap procedure for calculating the descriptive statistics used to represent pollutant EMC distributions in the water quality model. If the uncensored data were determined to be lognormally distributed with less than 50% of the data below the detection limit (censored), the bootstrap procedure was coupled with lognormal theory (i.e., data were log transformed prior to the bootstrap analysis). Otherwise, the original data plus the estimates of the censored data were analyzed in arithmetic space to calculate the arithmetic mean and standard deviation. Table B-11 summarizes the lognormal descriptive statistics, and Table B-12 summarizes the resulting arithmetic means. The latter data represent the land use specific pollutant EMCs in the Monte Carlo water quality model.

Table B-11: Lognormal Statistics for Modeling Pollutants Concentrations from Land Uses.

Land Use		TSS	TP	NH3	NO3	NO2	TKN	Diss Cu	Tot Pb	Diss Zn	Cl
	Arithmetic Units	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	µg/L	µg/L	µg/L	mg/L
Commercial	Mean	3.966	-1.242	-0.832	-0.884	-2.721	0.711	2.210	1.292	4.778	3.043
	St. Dev	0.609	0.680	1.218	0.635	1.060	0.804	0.685	1.389	0.703	1.226
Education	Mean	4.097	-1.375	-1.838	-0.750	-3.127	0.296	2.163	0.777	4.121	2.380
	St. Dev	0.923	0.515	1.111	0.626	1.177	0.604	0.733	0.891	0.531	1.264
Light Industrial	Mean	5.019	-1.328	-1.065	-0.574	-2.650	0.783	2.344	1.994	5.591	2.238
	St. Dev	0.741	0.828	0.957	0.828	0.667	0.694	0.764	1.041	0.769	0.590
Transportation	Mean	3.935	-1.229	-1.271	-0.687	-3.011	0.345	2.806	1.902	4.783	1.261
	St. Dev	0.834	0.992	0.608	0.749	1.056	0.654	1.116	0.631	1.040	0.998
Multi-Family Residential	Mean	3.144	-1.788	-1.208	-0.180	-2.932	0.346	1.768	0.812	3.965	2.124
	St. Dev	0.920	0.728	0.886	0.930	1.102	0.556	0.576	0.985	0.707	1.119
Single Family Residential	Mean	4.178	-1.170	-1.248	-1.219	-3.198	0.734	1.869	1.762	2.392	1.440
	St. Dev	1.026	0.640	0.964	1.274	1.191	0.747	0.783	0.997	1.085	0.570
Agriculture (Ventura County)	Mean	6.754	0.990	0.338	2.519	-2.120	1.948	2.839	3.015	3.252	3.666
	St. Dev	0.551	0.469	0.712	0.460	0.000	0.380	0.536	0.763	0.847	0.689
Vacant / Open Space	Mean	3.342	-3.060	-3.075	-0.033	-3.976	-0.458	-2.573	-1.246	1.293 [1]	1.864
	St. Dev	1.859	1.064	0.811	0.548	0.459	0.784	1.505	1.616	1.312	0.226

1 – Dissolved zinc for open space was estimated from the total zinc analysis of LACDPW monitoring data. Four data points for dissolved and total zinc from the National Stormwater Quality Database gave an average ration of dissolved to total zinc of 50 percent. For the open space land uses the variation of dissolved zinc was assumed to equal that of total zinc (i.e. same standard deviation) and the lognormal mean was set to give an average concentration of 8.6 ug/L for the open space land use, half of the average total zinc concentration of 17.2 ug/L.

Table B-12: Resulting Arithmetic Means from Lognormal Statistics for Modeling Pollutant Concentrations

Land Use	TSS	TP	NH3	NO3	NO2	TKN	Diss Cu	Tot Pb	Diss Zn	Cl
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	µg/L	µg/L	µg/L	mg/L
Commercial	63.5	0.364	0.913	0.505	0.115	2.81	11.5	9.55	152	44.5
Education	92.1	0.289	0.295	0.575	0.088	1.61	11.4	3.23	70.9	24.0
Light Industrial	199	0.373	0.545	0.793	0.088	2.78	13.9	12.6	360	11.2
Transportation	72.4	0.478	0.338	0.666	0.086	1.75	30.8	8.17	205	5.80
Multi-Family Residential	35.4	0.218	0.442	1.29	0.098	1.65	6.92	3.66	67.7	15.6
Single Family Residential	110	0.381	0.457	0.665	0.083	2.75	8.81	9.57	19.7	4.97
Agriculture (Ventura County)	998	3.00	1.81	13.8	0.120	7.54	19.7	27.3	37.0	49.6
Vacant / Open Space	159	0.083	0.064	1.12	0.021	0.860	0.237	1.06	8.61	6.62

B.2.5. Estimate of BMP Performance Parameters

BMP performance is a function of three factors: (1) the fraction of stormwater runoff receiving treatment (often referred to as percent of runoff captured, or simply percent capture); (2) the pollutant removal achieved in the unit by virtue of infiltration and/or evapotranspiration (generically referred to as volume reduction); and (3) the pollutant removal achieved in the treatment unit by virtue of improved water quality.

Capture efficiency calculations used to estimate results for the individual storms and volume reduction estimates are discussed in Section B.2.5.1. Pollutant removal estimates are described in Section B.2.5.2.

B.2.5.1. BMP Capture Efficiency

The modeled structural BMPs were analyzed as flow or volume-based. Different methods were used to calculate the capture efficiency of each type of BMP as discussed below.

B.2.5.1.1. Volume-based BMP Capture Efficiency

The GeoSYNOP program provides descriptive statistics of storm events based upon analysis of hourly rainfall records. Included in these statistics is the dry time between storms. This information, along with the storm depths and drainage rates of the volume based BMPs, was used to estimate the percent capture of the volume-based BMPs for each storm in the period of record. The percent capture calculations for volume-based BMPs required the following steps.

Step 1 – Estimate Runoff Volumes for Each Storm in the Period of Record Modeled

The runoff volume for each storm in the period of record (538 storms) was calculated for the tributary area draining to each BMP.

Step 2 – Determine the BMP Storage Capacity

Next, the available storage capacity of the BMP was calculated for each storm. If the time from the preceding storm was equal to or larger than the drawdown time of the BMP (48 hours for DEDBs), then the BMP was considered empty at the time of the storm.

If the time between storms was less than the drawdown time, then the capture volume was calculated to account for the size of the previous storm and the dry period between storms. This is done in order to account for insufficient time for the BMPs to completely empty before the next storm arrived. If the volume of stormwater runoff to the BMP from the previous storm was larger than the storage capacity of the BMP, then the BMP was assumed to have filled completely and the initial storage capacity (*ISC*) in equation 4 is zero.

If the runoff volume (for a storm occurring less than the drawdown time prior to the storm of interest) was less than the storage capacity of the BMP, then the difference between the storage capacity of the BMP and the runoff volume from the previous storm was considered available to capture runoff from the next storm. This volume is then added to the storage capacity created from outflow from the basin during the time of the storms as shown in equation 4.

$$TC = ISC + [BV \times DD \times T] \quad (4)$$

Where:

- TC = the treatment capacity (ft^3) of a volume-based BMP available to capture runoff over the duration of a storm
- ISC = the remaining storage volume after previous event (ft^3), initial storage capacity for storm of interest
- BV = the BMP volume (ft^3)
- DD = the draw down rate of a volume-based BMP in percent per hour (hr^{-1}) [2.08% per hour for a 48 hour draw down time]
- T = the storm duration (hr)

The above equation accounts for storage capacity that is created during emptying of the BMP while a storm occurs. That is, during long duration storms more runoff can be processed through the BMP than for a short storm of comparable rainfall intensities and runoff rates.

Step 3 – Determine BMP Percent Captures for Storms

The storage capacity estimated from step 2 is compared to the runoff volume estimate from step 1. If the storage capacity exceeds the storm runoff volume then the storm is considered to be completely (100%) captured. If the storage capacity is less than the runoff volume a volume of runoff equal to the storage capacity is considered treated by the BMP. The excess volume is assumed to bypass the BMP and enter the receiving water untreated.

B.2.5.1.2. Flow Based BMP Capture Efficiency

Vegetated swales represent the type of flow based BMP implemented as part of the Landmark Village Project. Capture efficiency for flow based BMPs depends on whether the BMP is on-line or off-line. On-line BMPs are designed without bypass capacity. Off-line BMPs are designed with a diversion structure for flows up to the treatment capacity.

Step 1 – Estimate the Depth of Runoff Captured on an Hourly Basis

The percent capture estimate for each storm is made through comparison of the hourly rainfall data comprising the storm event to the design rainfall intensity of the flow-based BMP. For on-line BMPs, if the depth of rainfall for a given hour exceeds the design rainfall intensity, then no treatment is credited for that hour. For off-line BMPs, if the depth of rainfall for a given hour exceeds the design rainfall intensity, then no treatment is credited for the rainfall above the design intensity (0.3 inches per hour). If the design capacity (in inches per hour) of the BMP

meets or exceeds the depth of rainfall occurring in a given hour, then all of the resulting runoff during that hour is considered treated.

Step 2 – Sum the Depth of Rainfall Capture for Each Storm Event

The depth of rainfall captured for each hour of rainfall during the storm event is then summed to give the total depth of rainfall considered captured by the BMP for the storm of interest.

Step 3 – Calculate the Percent Capture for Each Storm Event

The depth of rainfall captured during a given storm event is divided by the total depth of the storm to give the percent capture for the storm event that is used in the water quality model input.

Note that because flow-based BMPs are designed based on rainfall intensity and because a non-variable runoff coefficient method is used to convert rainfall to runoff over each catchment, the runoff characteristics of the catchment do not need to be known to calculate capture efficiency at the design stage. Rather, capture efficiency is based on comparison to design rainfall intensity to measured rainfall intensity.

B.2.5.1.3. BMP Capture Efficiency Results

The estimated average capture efficiencies for the structural BMPs in the Landmark Village treatment system are shown in Table B-13 for swales, Table B-14 for bioretention BMPs, and Table B-15 for the extended detention basin. The capture efficiency methods described above were used to estimate the fraction of runoff captured by each BMP type for each storm in the period of record.

Table B-13: BMP Percent Capture Estimate for Swales

Sizing Method	Design Precipitation Intensity (in/hr)	Estimated Capture Efficiency (%)[1]	Modeled Volume Reduction (%)
flow	0.30	80	25

1 – Capture efficiency was calculated with hourly rainfall data for each storm as described above and reported as an annual average.

Table B-14: BMP Percent Capture Estimate for Bioretention Areas

Sizing Method	Design Depth (in)	Estimated Capture Efficiency (%)[1]	Modeled Volume Reduction (%)
volume	Varies with imperviousness	80	25

1 – The bioretention BMPs will be sized to capture and treat 80 percent of the stormwater runoff on an average annual basis. The volume reduction, on an average annual basis, was modeled as equivalent to swales.

Table B-15: BMP Percent Capture Estimate for the Extended Detention Basin

Sizing Method	Tributary Area (ac)[1]	% Impervious	Runoff Coefficient	Estimated Capture Efficiency (%) [1]	Modeled Volume Reduction (%)
volume	10.9	84	0.90	80	15

1 – The basin will be sized to capture and treat 80 percent of the stormwater runoff on an average annual basis. Continuous simulation modeling indicated a storage volume of approximately 1.1 acre-feet would be required for a design emptying time of 48 hours. If additional storage capacity is available in the water quality basin above that required for the current tributary area, additional areas may be routed to the basin to utilize the treatment capacity.

As Table B-13, Table B-14 and Table B-15 show, volume and flow-based treatment BMPs will be sized such that an overall capture efficiency of 80 percent or greater is achieved for the treatment of stormwater runoff from the Landmark Village Project on an average annual basis. Capture efficiency in the water quality analysis was conservatively modeled as 80 percent on an average annual basis although off-line swales are expected to exceed this value. In areas where sufficient space is available, volume-based BMPs may be 'oversized' and exceed the treatment goal of 80 percent capture.

B.2.5.1.4. BMP Volume Reductions

The volume reduction achieved by a BMP is a function of the capture efficiency and the fraction of captured stormwater runoff that is infiltrated, evaporated, or transpired by vegetation.

BMPs specifically designed to infiltrate stormwater runoff are not included in the stormwater management system. However, data in the International BMP Database have shown that as much as 30 percent of stormwater volume captured by dry extended detention basins and 35 percent captured by swales can be lost to infiltration (Strecker et al., 2004), which indicates that this may be an important mechanism that should be included in the water quality analysis. Volume reductions achieved were conservatively modeled as 10 percent for the detention basin[1] and 25 percent for swale and bioretention BMPs. Bioretention areas for the Landmark Village Project will use underdrains to minimize the required footprint area; therefore, volume reduction is modeled conservatively as equivalent to swales.

B.2.5.2. BMP Pollutant Removal

Various data sources were examined to estimate the anticipated performance of the treatment BMPs. A comprehensive source of BMP performance information is the American Society of Civil Engineers (ASCE) International Stormwater BMP Database (ASCE/EPA, 2003, Strecker et al., 2001). The ASCE BMP database is comprised of carefully examined data from a peer-reviewed collection of studies that have monitored the effectiveness of a variety of BMPs in

[1] The average volume reduction was modeled as 10 percent as the basin will be lined to reduce infiltration and protect bank stability at the edge of the project in turn limiting the volumetric reduction of stormwater runoff.

treating water quality pollutants for a variety of land use types. The mean effluent water quality for treatment BMPs used for modeling purposes was based on values found in the International Stormwater BMP Database (ASCE/EPA, 2003). Recent work in characterizing BMP performance suggests that effluent quality rather than percent removal is more reliable in modeling stormwater treatment (Strecker et al. 2001).

To match site conditions, the BMP database studies were screened to exclude studies where BMP design or function was believed to result in significantly lower performance than the BMP design criteria that will be met for the Landmark Village BMPs. For example some of the detention basins studies had significantly lower maximum detention times than the 48 hour criteria for the water quality basins. The water quality data for detention basins with a drawdown time of less than 9 hours were excluded from the data set used to predict detention basin performance. Certain studies in the detention basins category were not considered comparable in function to the dry-extended detention basin that will be incorporated into the Landmark Village treatment system. Detention basins that were listed as either underground vaults or settling chambers were also excluded. All biofiltration (i.e. vegetated swales and filter strips) studies in the BMP database were deemed valid and were used in statistical analysis.

As with the estimation of land use EMCs, final effluent values to be used in modeling analysis were determined using a combination of regression-on-order statistics and the "bootstrap" method (see Section B.2.4.3).

Once the BMP sites had been screened for design criteria, the normality and lognormality of all BMP effluent sample data sets were tested using the Shapiro-Wilk goodness-of-fit test (Royston 1992). The majority of the pollutant data fit a lognormal distribution. The data that did not statistically fit the lognormal distribution were more closely approximated with a lognormal distribution than a normal distribution. The bootstrap method was applied differently depending on the distributional fit of the data. If the data fit a lognormal distribution, the log-transformed data were bootstrapped and an estimate of the mean and standard deviation were obtained in log space and then converted to arithmetic space. The assumption of lognormality was more stringently applied than normal by using an alpha significance value of 0.1. This was done to improve the estimate of the standard deviation when the assumption of lognormality fails. When analyzing data in log space there is a tendency to overestimate the standard deviation for relatively symmetric data and underestimate the standard deviation for severely skewed data. For datasets that did not fit the lognormal distribution, the raw data were bootstrapped to obtain mean and standard deviation values. Bootstrapping the data in arithmetic space assumes no distribution in those instances when a distribution could not be confirmed through goodness-of-fit testing.

Table B-16 shows the lognormal effluent quality descriptive statistics for detention basins and swales. These values were estimated using the above procedure on the ASCE/USEPA International BMP Database data (ASCE/EPA, 2003). Note that data were not available for nitrite or ammonia for detention basins. Removal was not simulated for these pollutants in the detention basin. Chloride removal was not simulated in the treatment BMPs as chloride is highly water soluble and is not a nutrient given to uptake by vegetation.

Table B-16: Summary of Lognormal Effluent Quality Statistics & Arithmetic Mean Effluent Quality for Modeled BMPs.

Pollutant	Lognormal Modeling Parameters				Arithmetic Means	
	Detention Basins		Swales & Bioretention		Detention Basins	Swales & Bioretention
	Mean	St Dev	Mean	St. Dev		
TSS	3.503	0.709	3.089	0.821	42.7	30.7
Total P	-1.262	0.553	-1.340	1.051	0.330	0.455
NH_3	NA	NA	-3.363	1.064	NA	0.061
NO_3	-0.346	0.671	-1.394	1.108	0.886	0.459
NO_2	NA	NA	-5.028	1.311	NA	0.015
TKN	0.460	0.522	0.336	0.593	1.81	1.67
Dissolved Cu	2.427	0.501	1.756	0.776	12.8	7.82
Total Pb	3.000	0.931	1.402	1.314	31.0	9.64
Dissolved Zn	3.786	0.705	3.231	0.714	56.5	32.6

NA - not available

B.2.6. Model Parameter Reliability & Assumptions

The input parameters for the water quality model fall into the following five main categories:

- Rainfall data;
- Runoff Coefficients;
- Land Use data;
- Stormwater pollutant EMCs; and
- BMP performance estimates.

Each of the categories of input data is evaluated for accuracy in reflecting the project site conditions:

Rainfall Data: A limited period of record (about 12 years of hourly data) is available from the Castaic Junction gauge monitored by the LACDPW. The Castaic Junction gauge is nearer to the project site and consistently measures precipitation amounts lower than recorded at the Newhall gauge. However, the limited period of hourly data collected at the Castaic Gauge is insufficient for water quality modeling and the rainfall data collected at the Newhall gauge was used. The rainfall data from the Newhall gauge are believed to overestimate the average annual rainfall by about 3 inches per year resulting in a conservative estimate of stormwater runoff volumes and changes in average annual volumes resulting from development. The San Fernando gauge which was used to fill in missing periods in the Newhall gauge measures only slightly lower average rainfall depths than the Newhall gauge and the data used from this gauge were corrected to account for this small difference. Thus the use of San Fernando gauge data to fill gaps in the Newhall record results in a more accurate representation of actual rainfall and does not significantly bias estimates of runoff volume or concentration.

Runoff Coefficients: The estimation of runoff coefficients, described in Section B.2.2, is highly dependant on soil properties (i.e. infiltration potential) and less dependent on parameters such as ET rates, slopes, and surface roughness. Soil properties are estimated as accurately as possible from available data such as soil surveys and site specific geomorphology studies. The result is estimates for runoff coefficients that may somewhat overestimate or underestimate stormwater runoff. The net result on the water quality model is that this parameter is not conservatively estimated; however, it is estimated as accurately as the available information permits. When combined with the overestimate of average annual rainfall and land use percent impervious values (discussed below), stormwater runoff volumes are somewhat conservatively predicted.

Land Use Data: Land use data is generally considered a relatively accurate and quantifiable input parameter. The land use data for the developed conditions can be used to classify land use type and compute area. The percent impervious values used in the water quality model for the urban land uses in the developed project condition are based upon the values listed in the LA County Hydrology Manual (2006). The percent impervious values assigned to types of urban land uses may slightly overestimate imperviousness for some land uses because the Manual is intended for drainage and flood control analysis of large storm events. However on a whole the Hydrology Manual values are generally considered to be a fairly accurate quantification of impervious where detailed site designs are not available. The emphasis of modeling efforts described herein is to quantify imperviousness as accurately as possible without intentionally incorporating conservatism.

Stormwater Pollutant EMCs: Stormwater pollutant EMCs are estimated from monitoring data collected by the LACDPW from land use characterization stations and generally do not have site design and source control BMPs that will be implemented for the Landmark Village Project. Therefore the stormwater pollutant EMCs estimated from the LACDPW data are probably slightly conservative compared to the pollutant concentrations in stormwater runoff that will occur from the developed conditions of the project site.

BMP Capture Efficiency & Effluent Concentrations: Stormwater capture efficiency estimates were calculated in Excel spreadsheets to provide results on a storm-by-storm basis for input into the water quality model, to accurately reflect the anticipated performance of the structural stormwater BMPs.

BMP effluent concentrations are based on studies contained in the International BMP database. These studies are screened to remove data for undersized (i.e. inadequate design criteria) BMPs that are likely to have pollutant removal performance substantially less than the BMPs to be constructed for Landmark Village. This screening is believed to improve the accuracy of BMP performance estimates; however, it is only intended to remove BMPs that are clearly unrepresentative in terms of sizing. The screening process is intended to include BMPs with adequate performance that may not be as well designed or maintained as the structural BMPs that will be part of Landmark Village. It is anticipated that the BMPs for the Landmark Village Project will perform as well, if not slightly better than the projected performance based on the database. A major issue in the use of the International Database is representativeness for semi-

arid climates. In this respect the database contains sites from different climates, but does include a number of sites from semi-arid climates, including data for over 40 sites studied by Caltrans.

Conclusions: The runoff coefficient, land use type and area, land use percent imperviousness and BMP performance model input parameters are thought to be reasonably accurate representations of the site conditions and do not increase the conservativeness of the water quality model. The rainfall data and stormwater pollutant EMC estimates are believed to result in conservative estimates of stormwater runoff volumes, pollutant concentrations and therefore pollutant loads. Overall, the predevelopment model input parameters likely result in a slight underestimation of estimated loads and concentrations in the existing condition. The water quality estimates for the developed project condition are also believed to be conservative (i.e., tend to overestimate loads and concentrations) due to pollutant concentration estimates, and BMP performance estimates that in general do not include the benefits of site design or source control BMPs that are planned to be implemented in Landmark Village.

B.3. Model Methodology

A Monte Carlo simulation method was used to develop the statistical description for storm water quality. In this approach, the storm water characteristics from a single rainfall event are first estimated. The rainfall depth was determined by randomly sampling from the historical rainfall depth frequency distribution. Similarly, an EMC was determined by randomly sampling from the frequency distribution of EMCs. The rainfall volume and EMC were used to determine runoff volume, pollutant concentration, and pollutant load of the single storm event. BMP volume reduction and performance (effluent quality), determined by randomly sampling from the developed frequency distributions, were used to calculate the pollutant removal resulting from treatment in the BMP system. This procedure was then repeated thousands of times (20,000), recording the volume, EMC and load from each randomly selected storm event, including treatment for the developed project condition. The statistics of these recorded results provide a description of the average characteristics and variability of the volume and water quality of storm water runoff.

- Total Suspended Solids (sediment)
- Total Phosphorus
- Ammonia
- Nitrate
- Nitrite
- Total Nitrogen[2]
- Dissolved Copper
- Total Lead
- Dissolved Zinc
- Chloride

[2] TKN is modeled, but the results are not reported. Total Nitrogen results are reported from the sum of nitrate, nitrite, and TKN.

The steps in the Monte Carlo Water Quality Model are as follows:

1. Develop a statistical description of the number of storm events per year, and randomly select a number N_{storms}.
2. Estimate the volume of storm runoff for each land use area from a randomly selected storm event.
3. Randomly select a pollutant concentration in storm runoff for each land-use area and each pollutant.
4. Calculate the total runoff volume, pollutant load, and concentration in runoff from the modeled portion of the project, for both existing and developed conditions.
5. Calculate a total annual pollutant load by repeating steps 2-4 N_{storms} times, where N_{storms} is the number of storms per year, randomly selected in step 1.
6. Repeat steps 1 - 6 a total of 20,000 times for each pollutant modeled, recording the estimated pollutant concentration and annual load for each iteration.
7. Develop a statistical representation (mean annual value) of the recorded storm water pollutant loads and concentrations.

Each of the seven steps is described below.

B.3.1. Storms & Stormwater Runoff (steps 1 & 2)

Step 1 – Statistical Representation of Number of Storm Events per Year

Number of Storms per Year

The number of storm events per year was calculated for the 35 complete years in the available period of record from 1969 – 2003. The modeled average number of storm events per year (> 0.1 inches) was 15.4, with a standard deviation of 6.2. Figure B-11 illustrates a frequency histogram of the number of storm events per year at the Newhall gauge. The number of storm events per year was modeled with a normal distribution. In the simulation, the number of storms per year was determined by randomly sampling from the normal distribution and rounding to the nearest whole number, using the equation:

$$N_{storms} = 15.4 + 6.2\ R_N$$

where:

R_N = a standard normal variant with a mean of 0 and a standard deviation of 1.

If the arbitrary number of storms per year was zero or negative, then the normal distribution was re-sampled until a positive number was obtained.

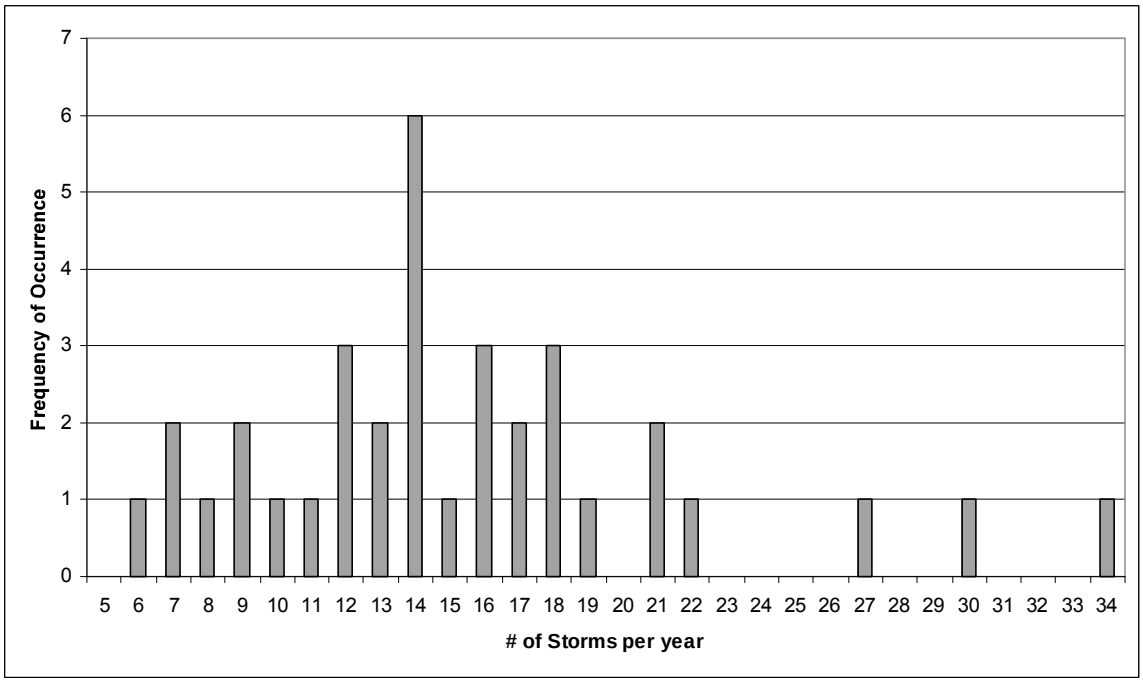

Figure B-11: Distribution of Storms per Year at the Newhall Gauge

Step 2 – Estimate the Volume of Storm Runoff from a Storm Event.

The runoff volume from each storm was estimated using the following equation:

$$V=R_v PA \quad (5)$$

where:

V = the stormwater runoff volume (ft^3)

P = the rainfall depth of the storm (ft)

A = the drainage area (ft^2)

R_v = the mean volumetric runoff coefficient, a unit-less value that is a function of the imperviousness of the drainage.

For sub-basins that contain multiple land-use types, the total stormwater runoff volume is determined as the sum of runoff from each land-use type:

$$V_{wshed} = \Sigma_{lu}\, V_{lu} = \Sigma_{lu}\, (R_{v\,lu}\, PA_{lu}) \quad (6)$$

where *lu* designates the land-use type. It is assumed that rain falls uniformly over all land-uses in the sub-basin.

The steps used to calculate the volume of runoff from a randomly selected storm event were:

Step 2a Obtain a rainfall depth by randomly sampling from the 538 storm events.

Step 2b For each land-use area calculate a runoff volume using equation (5). The same rainfall depth is applied to each land-use area.

Step 2c Sum the runoff volumes from each land-use area to obtain the total runoff from the watershed for a particular storm event with equation (6).

B.3.2. Pollutant Loads & Concentrations (step 3 & 4)

Step 3 – Estimate a Pollutant Concentration in Storm Runoff from Each Land Use Area

Runoff Concentration

The distribution of land use-based pollutant concentration in storm runoff was developed based on the process described in Section B.2.4.3. For each storm event, stormwater EMCs were sampled randomly for each modeled land use and water quality parameter. The runoff concentration from each land-use area was evaluated with the expression:

$$C_{land-use} = \exp\left(\mu_{\ln x} + \sigma_{\ln x} R_N\right) \quad (7)$$

where:

$\mu_{\ln x}$ = the log-normal mean

$\sigma_{\ln x}$ = the log-normal standard deviation

R_N = a standard normal random variable

Step 4 – Calculate the Total Runoff Volume, Pollutant Load, and Pollutant Concentration in a Storm Event

Step 4A - The total runoff volume in the watershed was calculated with equation (6) as discussed in Step 2:

$$V_{wshed} = V_{land-use1} + V_{land-use2} + \ldots + V_{land-usei} \quad (8)$$

where the same randomly selected rainfall event was used to calculate runoff volume in each of the land-use areas.

Step 4B - The total pollutant load from the watershed was calculated by:

$$L_{wshed} = V_{land-use1} C_{land-use1} + \ldots + V_{land-usei} C_{land-usei} \quad (9)$$

where the concentration in each individual land-use area was calculated with equation (7) discussed in step 3.

Step 4C - The average pollutant concentration in runoff from the entire watershed from a single storm event was calculated by dividing the total watershed load (Step 4B) by the total watershed runoff volume (Step 4A):

$$C_{wshed} = L_{wshed} / V_{wshed} \quad (10)$$

Model steps up to 4C (Eq 10) were used in the model calculations for catchments with and without modeled BMPs. The resulting values from Equation 9 and Equation 10 represent the

end model output for catchments without modeled BMPs and represent intermediate calculations for catchments with modeled BMPs

Catchments with treatment BMPs used additional calculations to determine the reduction in pollutant load and concentration achieved with treatment BMPs. The fraction of stormwater runoff receiving treatment was calculated for each storm event, using the capture efficiency associated with that event, as described in Section B.2.5. BMP performance was modeled using a randomly selected effluent concentration achieved within the BMP for each water quality pollutant.

Step 4D - The total pollutant load from watersheds with treatment BMPs was calculated by:

$$L_{wshed\_BMPs} = \left[Cap_{\%} \times V_{wshed} \times C_{eff} \times (1 - VR\%)\right] + \left[(1 - Cap_{\%}) \times V_{wshed} \times C_{wshed}\right] \quad (11)$$

where:

$Cap_{\%}$ is the volumetric percent capture of the BMP.

C_{eff} is the randomly determined effluent concentration from the BMP. C_{eff} was determined from sampling from the lognormal distribution described by the parameters contained in Table B-16.

$VR\%$ is the percent reduction in effluent volume achieved by the BMP (see Section B.2.5.1.3).

V_{wshed} and C_{wshed} were calculated per Steps 4A and 4C, respectively

Step 4E - The average pollutant concentration in runoff from the entire watershed with treatment from a single storm event was calculated by dividing the total watershed load with treatment by the total watershed runoff volume less the volume lost in BMPs:

$$C_{wshed\_BMPs} = L_{wshed\_BMPs} / V_{wshed\_BMPs} \quad (12)$$

where:

$$V_{wshed\_BMPs} = V_{wshed} \times \left[1 - (Cap_{\%} \times VR\%)\right] \quad (13)$$

The results of step 4D (Eq 11) and step 4E (Eq. 12) were used to compute model results for developed conditions with treatment.

Figure B-12 provides a diagrammatic representation of these water quality calculations.

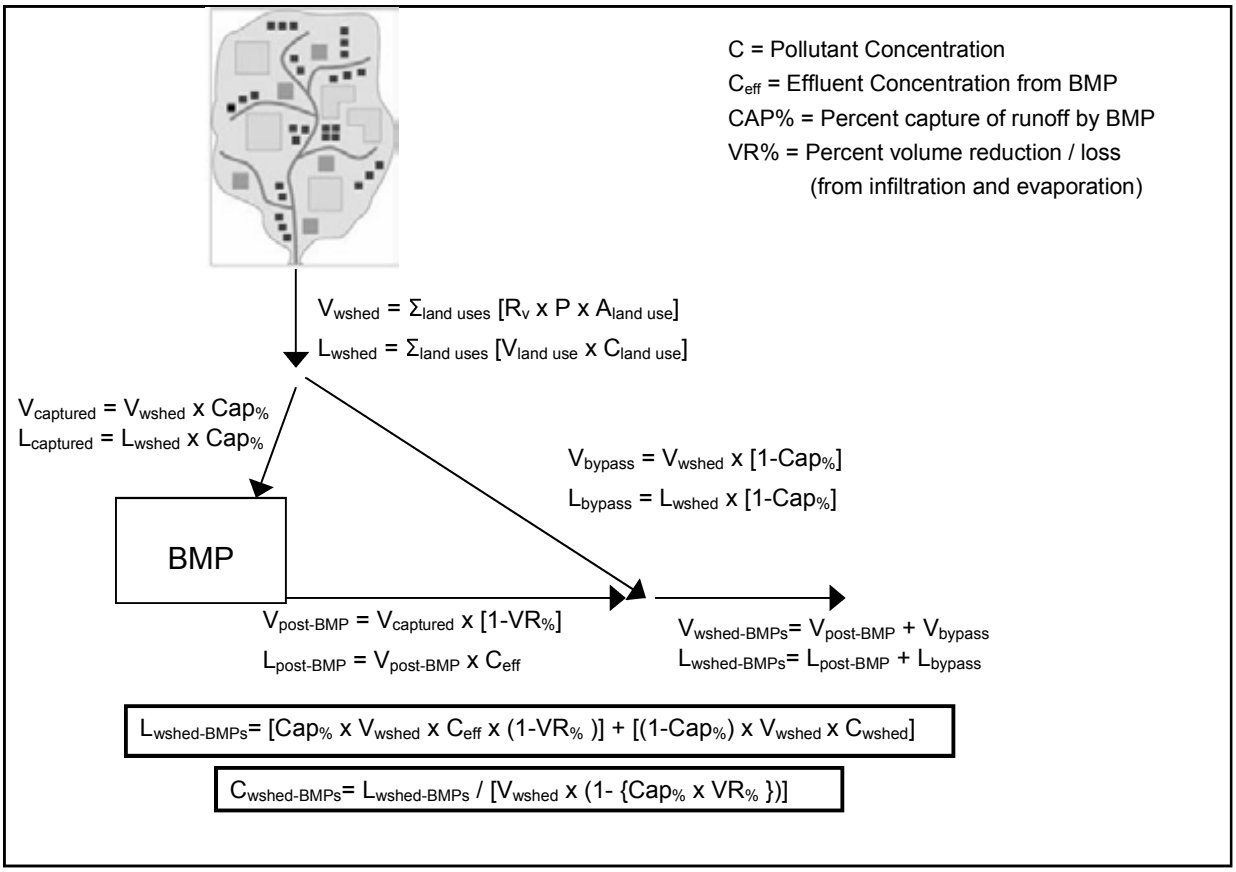

Figure B-12: Diagrammatic representation of water quality calculations.

B.3.3. Annual Pollutant Loads, Concentrations, and Distributions (steps 5, 6, & 7)

Step 5 – Calculate a Total Annual Pollutant Load

The annual pollutant load is simply the sum of pollutant loads generated from all storms in a given year, based on the random selection described in Step 1. Therefore, steps 2-4 were repeated N_{storms} times (where N_{storms} was randomly selected per step 1), recording the total pollutant load from each randomly selected storm event. The individual storm loads were summed to obtain the total annual pollutant load.

Step 6 & 7 – Determine Distribution of Storm Concentration and Annual Loads

Steps 1-5 were repeated a total of 20,000 times, recording the pollutant concentration and annual load from each iteration. The resultant distributions can be used to present a frequency distribution for pollutant concentrations or loads using statistics calculated from the 20,000 Monte-Carlo iterations.

B.3.4. Model Methodology Assumptions

The following five key assumptions are made for the Monte Carlo water quality modeling methodology:

1. The assumed probability distributions of model parameters;
2. The assumption of independence between model parameters (i.e. no correlation between randomly determined variables);
3. Assigning a Lower Limit to BMP Effluent Concentrations;
4. Limiting pollutant removals to pollutants with data; and
5. Modeling structural BMPs to only remove pollutants and not acting as a source.

The implications of each of these assumptions to the water quality projections are discussed below.

1) Distribution Assumptions: Probability distributions are assumed to represent the number of storms per year, stormwater pollutant concentrations, and BMP effluent concentrations. Observed rainfall data (i.e., storm frequency) and stormwater monitoring data are fit with either a normal or lognormal distribution using standard statistical procedures. The values of storms per year, rainfall depth, runoff pollutant concentration, and BMP effluent concentrations used in given iteration in the Monte Carlo analysis are governed by the selected distributions. Large samples of these estimated variables will approximate the assumed distributions, and will have the same mean and variance that was observed in the rainfall and monitoring data. The following describes the distributions for various input parameters.

Storms per Year: Figure B-11 shows the number of storms per year occurring at the Newhall rain gauge (augmented with data from the San Fernando gauge). The number of storms occurring per year at the Newhall gauge appears to lie between the normal and lognormal distributions. The normal distribution was used to determine the number of storms per year simulated in the water quality model, as use of the lognormal distribution would overestimate the average annual rainfall, as well as its variability, when the distribution of the data is not heavily skewed. As discussed in Section B.2.6, use of rainfall data collected at the Newhall gauge already tends to overestimate the average annual rainfall for the Project site. When using the normal distribution to randomly determine the number of storm per year, the resulting average annual rainfall output from the water quality model is typically in the range of 17.9 to 18.0 inches per year. This is in close agreement with the average annual rainfall from runoff producing storms of 17.9 inches determined directly from the rainfall data (see Table B-1).

Stormwater Pollutant Concentrations: The Shapiro-Wilk Test was used to determine the statistical distribution that best represents the raw stormwater runoff monitoring data collected in Los Angeles and Ventura Counties. In most instances the data were found to be log-normally distributed at a confidence level of 0.10. In some instances, the data were not well fit by either the normal or lognormal distributions, but were found to be more closely approximated by the log-normal distribution. For data sets with greater than 50 percent non-detects or that were not log-normally distributed according to the Shapiro-Wilk test, data were analyzed (ROS and

bootstrap) in arithmetic space as to not unreasonably overestimate the standard deviation of the data set. Since stormwater pollutant concentrations, in general, tend to be well approximated by the lognormal distribution (Helsel and Hirsh, 2002), the data sets that did not meet the lognormal criterion are still believed to belong to a log-normally distributed population, but the number of data points is too few to statistically confirm that this is the case. Therefore, simulations of stormwater concentrations in the water quality model were still conducted in lognormal space. This assumption is believed to result in a more accurate prediction than would the application of the normal distribution.

BMP Effluent Concentrations: Goodness-of-fit tests conducted on the raw BMP effluent monitoring data from the International BMP Database with the Shapiro-Wilk Test either resulted in (1) confirmation of the appropriateness of the lognormal distribution for the data; or (2) in the instances when the data did not meet the significance criteria of a p value > 0.1, that the data were more closely approximated with the lognormal distribution than the normal. The use of the lognormal distribution to represent BMP effluent concentrations results in higher average estimates of BMP effluent concentration. This is believed to be a more accurate estimation of BMP performance than use of the normal distribution, and is considered a more conservative assumption (leading if anything to higher than anticipated effluent concentrations).

2) Assumption of No Correlation between Model Parameters: The water quality model randomly samples for stormwater pollutant concentrations independent of the storm depth or antecedent dry period. The validity of this assumption is supported by analyses conducted by Environmental Defense Sciences (2002) who did not find a strong correlation between rainfall volume and event mean concentrations (EMCs) in the LA County data for the education land-use site. Data analyses for the single family residential land use were found to be weakly correlated (R^2 of 0.6 ± 0.1) for some pollutants with storm depth; however some pollutant showed little correlation between these variables. Where weak correlations were present, stormwater pollutant concentrations decreased with storm size.

Correlations between pollutant concentration and antecedent dry period were similarly variable. For the single family land use correlations between pollutant concentration and antecedent dry period were moderately significant for a few pollutants (R^2 of 0.8 ± 0.03), and weak for other pollutants. Correlations between pollutant concentration and antecedent dry period varied widely for the educational and multi-family land uses.

The results of these analyses indicated that no consistent level of correlation was determined between the stormwater EMCs and the rainfall depth or the antecedent dry period where a significant correlation was found to exist; most pollutants and land-uses showed weak correlations or no correlation. On this basis, stormwater pollutant concentrations are sampled independent of storm depth and antecedent dry period in the water quality model.

Effluent concentrations are considered more reliable estimator of treatment performance than percent removal (Strecker et al. 2001). BMP effluent concentrations were sampled independently of stormwater concentrations (i.e. influent concentration to the BMP) in the water

quality model. As with the pollutant EMCs, independent sampling of effluent concentrations preserves the mean and standard deviation in the monitoring data.

3) BMP Performance – Irreducible Pollutant Effluent Concentrations: When sampling from the lognormal distribution to estimate BMP performance with an effluent concentration it is possible to select values approaching or equal to zero. While well functioning BMPs are capable of achieving high rates of pollutant removal, it is generally accepted that BMPs cannot completely remove pollutants from the water column. In effect BMPs, at best, can achieve what is called an "irreducible pollutant concentration" (Schueler, 1996). In an effort to prevent overestimating BMP performance in the model, lower limits were set for the effluent concentrations of each modeled pollutant and BMP. The lowest observed effluent value in each pollutant data set was used as the irreducible pollutant effluent concentration in the water quality model.

4) BMP Performance – Limiting Pollutant Removal Estimates to Available Data: Table B-17 presents model parameters for estimating BMP pollutant effluent concentrations. Pollutant removal is only simulated for those pollutants with available data from the International BMP Database. In instances where data is not available for a parameter, no treatment is assumed for that parameter. This does not prevent the model from calculating load reductions of the pollutant as a result of hydrologic source control.

5) BMP Performance – BMPs are not a Source of Pollutants: In instances when the randomly determined BMP effluent concentration exceeds the modeled influent concentration, no pollutant removal occurs and the effluent concentration is modified to equal the influent concentration. This prevents BMPs from acting as a source of pollutants in the water quality modeling. The commitment to regular and effective maintenance of the stormwater BMPs provides support for this assumption.

Conclusions: The above assumptions are expected to improve the accuracy of the water quality model estimates. The net result for the model outputs are somewhat conservative estimates of pollutant loads and concentrations due to estimation of model input parameters that are not compromised by the model methodology.

B.4. Model Reliability

Factors that affect model reliability include variability in environmental conditions and model error. To account for environmental variability, a statistical modeling approach was used that takes into account the observed variability in precipitation from storm to storm and from year to year. The model also takes into account the observed variability in water quality from storm to storm, and for different types of land uses. One way to express this variability is the coefficient of variation (COV) which is the ratio of the standard deviation of the variable to the mean value. Based on the statistical model, the range of COVs for pollutant loads was from 0.5 to 0.8 on an average annual basis, depending on the pollutant. This variability, or greater, is expected in typical storm water runoff.

Model error relates to the ability of the model to properly simulate the processes that affect storm water runoff, concentrations, and loads. Ideally model error is measured through calibration, but calibration is not feasible when considering a future condition. We are confident that the model is a reasonable reflection of storm water processes because the model relies largely on measured regional data. For example, the runoff water quality data are obtained from a comprehensive monitoring program conducted by LA County that has measured runoff concentrations from a variety of land use catchments and for a statistically reliable number of storm events. In addition parameter estimation is fairly conservative resulting in moderately conservative estimates of pollutant concentrations and loads.

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APPENDIX C
NEWHALL RANCH STORMWATER MONITORING DATA

APPENDIX C - NEWHALL RANCH STORMWATER MONITORING DATA

March 6, 2000

Newhall Ranch Monitoring Station	Hardness mg/L	Calcium mg/L	Magnesium mg/L	Potassium mg/L	Sodium mg/L	Alkalinity mg/L	Sulfate mg/L	Chloride mg/L	Nitrate mg/L	E.Coli MPN/100 mL	TDS mg/L
A-Mouth of Potrero	2360	324	378	30	1360	400	3690	780	16.1	8160	7530
B-Mouth of San Martinez	1070	229	122	8	392	210	1520	130	2.8	3090	2690
D-Mouth of Middle Canyon	44	11	4	6	9	30	16	3	12.4	133	160
E-Top of Chiquito Canyon	61	18	4	8	13	40	37	9	2.6	213	150

	Boron mg/L	Copper ug/L	Iron ug/L	Manganese ug/L	Zinc ug/L	Aluminum ug/L	Arsenic ug/L	Barium ug/L	Berylium ug/L	Cadmium ug/L	Chromium ug/L
A-Mouth of Potrero	2.6	20	4770	880	50	4570	5	155	0.6	0.4	7
B-Mouth of San Martinez	0.8	150	51500	4230	300	44000	21	391	7	8.8	47
D-Mouth of Middle Canyon		10	1290	350	30	2230		136	0.4	0.4	2
E-Top of Chiquito Canyon		40	11700	970	150	6280	3	210	1.4	1	10

	Lead ug/L	Mercury ug/L	Nickel ug/L	Selenium ug/L	Total Coliform MPN/100ml	Fecal Coliform MPN/100ml	TSS mg/L	VS mg/L	pH
A-Mouth of Potrero	8	0.01	22	12	50000	1600	1180	32800	8.2
B-Mouth of San Martinez	47.7	0.06	180	11	160000	1700	28000	40000	8
C-1/2 Mile Upstream of Onion Field					90000	11000			
D-Mouth of Middle Canyon	7.7	6			>160000	>160000	600	4100	7.5
E-Top of Chiquito Canyon	19.1		25		2400	2400	3490	9300	7.1

SS = suspended solids
VS = volatile solids

March 8, 2000

Newhall Ranch Monitoring Station	Hardness mg/L	Calcium mg/L	Magnesium mg/L	Potassium mg/L	Sodium mg/L	Alkalinity mg/L	Sulfate mg/L	Chloride mg/L	Nitrate mg/L	E.Coli MPN/100 mL	TDS mg/L
A-Mouth of Potrero	2090	266	347	39	1470	360	3700	960	18.8	6470	7230
B-Mouth of San Martinez	1340	304	142	10	413	210	1900	120	3.1	2430	2960
C-1/2 Mile Upstream of Onion Field	147	44	9	3	10	80	87	3	1.6	323	190
D-Mouth of Middle Canyon	73	21	5	6	10	40	17	3	18.1	162	160
E-Top of Chiquito Canyon	153	43	11	11	18	70	119	12	2.9	420	260

	Boron mg/L	Copper ug/L	Iron ug/L	Manganese ug/L	Zinc ug/L	Aluminum ug/L	Arsenic ug/L	Barium ug/L	Berylium ug/L	Cadmium ug/L	Chromium ug/L
A-Mouth of Potrero	2.9	10	2460	510	30	1580	5	94.4	0.3	0.2	4
B-Mouth of San Martinez	0.8	200	47500	5210	360	69700	27	573	20	13.6	70
C-1/2 Mile Upstream of Onion Field		170	44600	6950	330	85100	13	2360	14	2	39
D-Mouth of Middle Canyon			1510	300	30	2300		132	0.5	0.4	2
E-Top of Chiquito Canyon		100	30700	2110	300	2360	6	470	4.4	2.7	27

	Lead ug/L	Mercury ug/L	Nickel ug/L	Selenium ug/L	Total Coliform MPN/100ml	Fecal Coliform MPN/100ml	TSS mg/L	VS mg/L	pH	TOC mg/L	Diazinon ug/L
A-Mouth of Potrero	4.2	0.03	15	12	30000	7000	490	850	8.2	21.2	ND
B-Mouth of San Martinez	59.2	0.24	330	11	>160000	205	54200	1840	7.8	11.6	ND
C-1/2 Mile Upstream of Onion Field	95.2	0.45	103	4	160000	1600	36000	1460	8.1	9.4	4
D-Mouth of Middle Canyon	7.6	0.02	6		50000	2400	10700	160	7.9	4	ND
E-Top of Chiquito Canyon	54.5	0.14	64	2	>160000	160000	9800	750	8	15.5	

SS = suspended solids
VS = volatile solids

APPENDIX D
REVIEW OF BACTERIA DATA FROM SOUTHERN CALIFORNIA WATERSHEDS

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REVIEW OF BACTERIA DATA FROM SOUTHERN CALIFORNIA WATERSHEDS

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SUMMARY

Available data from Southern California watersheds demonstrate that both existing and EPA-recommended bacteria water quality criteria are routinely exceeded in fresh water creek and river flows, often by one or more orders of magnitude. Exceedances of criteria occur even for flows from largely natural, undeveloped watersheds with little human influence. Even in urbanized watersheds, there is strong evidence that the predominant source of indicator bacteria may be natural (not anthropogenic) – including, for example, bacteria from wildlife, birds, and regrowth within the environment, including sediments. Both measurement data and numerous literature sources have shown that both wet and dry weather bacteria concentrations frequently exceed objectives in creeks and rivers, and that bacteria concentrations rise dramatically during wet-weather periods.

Data from Orange County coastal watersheds indicate that although bacteria in storm water runoff may be elevated within urban storm drain systems, the level of development within these watersheds has little if any effect on the concentrations of indicator bacteria in the receiving waters. These results are consistent with data from other watersheds within Orange County and in other parts of Southern California. No clear trend is evident in bacteria concentrations over time, with concentrations remaining relatively steady, even in areas where land use characteristics have changed over time. Both the concentrations of bacteria in runoff and the impacts of elevated bacteria concentrations on downstream water quality appear to vary by site and with the size of the contributing stream, and thus are likely a function of the dominant sources of bacteria, local hydrologic conditions and climate, and other site-specific factors.

INTRODUCTION

Flow Science was retained by The Irvine Company to review available data and information on the concentrations of indicator bacteria in storm water and dry weather runoff. The goals of this study were to evaluate variations in the concentrations of bacteria during both wet and dry conditions, variations in bacteria levels with the level of development in a watershed or drainage area, changes in bacteria levels over time or with changes in development or land use areas, and the sources of bacteria in runoff and in receiving waters.

In conducting the analysis, Flow Science utilized water quality criteria and thresholds to evaluate available data. These thresholds were obtained from the Water Quality Control Plan (Basin Plan) for the Santa Ana Region, which contains fecal coliform water quality objectives for inland surface waters that apply to the beneficial uses of water contact recreation (REC-1)[1] and non-water contact recreation (REC-2)[2], from proposed EPA water quality criteria, and from Title 17 "beach posting" thresholds. These thresholds are discussed in greater detail below.

Flow Science evaluated data on bacteria concentrations in Southern California. Data were available for watersheds along the Newport Coast, for inland watersheds, and from Los Angeles County. In addition, Flow Science reviewed literature and studies conducted by others.

BACKGROUND: BACTERIA WATER QUALITY STANDARDS

The Basin Plan bacteria objectives currently contained in the Santa Ana Basin Plan were originally developed by the National Technical Advisory Committee (NTAC) to the Federal Water Pollution Control Administration in 1968.[3] These recommendations were based upon prospective

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[1] See Basin Plan at p. 4-6: "REC-1 Fecal coliform: log mean less than 200 organisms/100 mL based on five or more samples/30 day period, and not more than 10% of the samples exceed 400 organisms/100 mL for any 30-day period."

[2] See Basin Plan at p. 4-6: "REC-2 Fecal coliform: average less than 2000 organisms/100 mL and not more than 10% of samples exceed 4000 organisms/100 mL for any 30-day period."

[3] See *Water Quality Criteria, a Report of the National Technical Advisory Committee to the Secretary of the Interior*, Federal Water Pollution Control Administration: Washington, D.C., April 1, 1968, at p. 8 and p. 12:

"Surface waters should be suitable for use in "secondary contact" recreation – activities not involving significant risks of ingestion – without reference to official designation of recreation as a water use. For this purpose, in addition to aesthetic criteria, surface waters should be maintained in a condition to minimize potential health hazards by utilizing fecal coliform criteria. In the absence of local epidemiological experience, the Subcommittee recommends an average not exceeding 2,000 fecal coliforms per 100 ml and a maximum of 4,000 per 100 ml, except in specified mixing zones adjacent to outfalls."

epidemiological studies conducted by the United States Public Health Service in 1948, 1949, and 1950. These studies found an "epidemiologically detectable health effect" at levels of 2300 to 2400 coliforms per 100 ml at bathing beaches on Lake Michigan (at Chicago) and in the Ohio River. Later work conducted in the mid-1960s showed that approximately 18% of the coliforms present in the mid-1960s at the Ohio location belonged to the fecal coliform subgroup. The recreational contact water quality criteria suggested by the committee were based upon the fraction of coliforms present as fecal coliforms and a factor of safety of two.

The fecal coliform standards recommended in 1968 were adopted by many states and municipalities and remain in use in many locations (including in the Santa Ana Region). Several studies conducted since 1968 have questioned these criteria and recommended use of alternatives.[4] As early as 1972, a Committee formed by the National Academy of Science-National Academy of Engineers noted the deficiencies in the study design and data used to establish the recreational fecal coliform criteria, and stated that it could not recommend a recreational water quality criterion because of a paucity of valid epidemiological data (Committee on Water Quality Criteria, 1972).

In response to these concerns, EPA in 1972 initiated studies at marine and freshwater bathing beaches that were designed to correct the deficiencies in the earlier studies and analyses. These studies were conducted at sites contaminated either with pollution from multiple point sources (usually treated effluents that had been disinfected) or by effluents discharged from single point sources. The studies examined three bacterial indicators of fecal pollution (*E. coli*, enterococci, and fecal coliforms) and found that fecal coliform densities showed "little or no correlation" to gastrointestinal illness rates in swimmers. In contrast, a good correlation was found between swimming-associated gastrointestinal symptoms and either *E. coli* or enterococci in swimming waters (Dufour, 1984). Based on these studies, EPA in 1986 proposed section 304(a) criteria for full body contact recreation based upon *E. coli* and/or enterococci but noted that "it is not until their adoption as part of the State water quality standards that the criteria become regulatory" (USEPA, 1986).

EPA's current recommendations for bacteria water quality objectives (USEPA, 2003) include the use of *E. coli* and/or enterococci as the basis for water quality criteria to protect fresh recreational waters and the use of enterococci as the basis for marine water quality criteria. The EPA recommends that the use of fecal coliform be discontinued for both freshwater and marine

---

"Fecal coliforms should be used as the indicator organism for evaluating the microbiological suitability of recreation waters. As determined by multiple-tube fermentation or membrane filter procedures and based on a minimum of not less than five samples taken over not more than a 30-day period, the fecal coliform content of primary contact recreation waters shall not exceed a log mean of 200/100 ml, nor shall more than 10 percent of total samples during any 30-day period exceed 400/100 ml."

[4] For a summary of these studies, see the discussion provided on pages 1-3 of the *Ambient Water Quality Criteria for Bacteria – 1986*, USEPA 440/5-84-001, January 1986.

waters. EPA's recommendations recognize that bacteria concentrations are quite variable and are best characterized in terms of a probability distribution. Because bacteria concentrations tend to follow log-normal distributions, EPA's current recommendations specify that compliance should be based upon geometric means computed with data collected over a long-term (e.g., 30 days, or seasonally) and "upper percentile values," clarifying that compliance should not be determined using "single sample maximum" values. Upper percentile values are calculated bacteria densities that are intended to correspond to a known geometric mean-based risk level, and are intended to be used to interpret any single measurement. EPA recommends that states acquire enough sample data to calculate site-specific upper percentile values to characterize water quality for waters where exposure is greatest (e.g., bathing beaches). EPA's recommended water quality criteria for freshwater and marine waters are presented in Tables 1 and 2.

Table 1. Water quality criteria for bacteria recommended by EPA for fresh recreational waters

Risk level[a] [% of swimmers]	Geometric mean density [per 100 ml]	Upper Percentile Value Allowable Density [per 100 ml]			
		75th percentile	82nd percentile	90th percentile	95th percentile
Enterococci criteria					
0.8	33	62	79	107	151
0.9	42	79	100	137	193
1.0	54	101	128	175	247
E. coli criteria					
0.8	126	236	299	409	576
0.9	161	301	382	523	736
1.0	206	385	489	668	940

a) The risk level corresponds to the anticipated excess illness rate. For example, a risk level of 0.8% is believed to correspond to an illness rate of 8 gastrointestinal illnesses per 1,000 swimmers in excess of background illness rates.

Table 2. Water quality criteria for enterococci recommended by EPA for marine recreational waters

Risk level[a] [% of swimmers]	Geometric mean density [per 100 ml]	Upper Percentile Value Allowable Density [per 100 ml]			
		75th percentile	82nd percentile	90th percentile	95th percentile
0.8	4	13	20	35	63
0.9	5	16	24	42	76
1.0	6	19	29	50	91
1.1	8	23	35	61	110
1.2	9	28	42	73	133
1.3	11	34	51	89	161
1.4	14	41	62	107	195
1.5	17	49	75	130	235
1.6	20	60	91	157	284
1.7	24	72	109	189	344
1.8	29	87	132	229	415
1.9	35	105	160	276	502

a) The risk level corresponds to the anticipated excess illness rate. For example, a risk level of 0.8% is believed to correspond to an illness rate of 8 gastrointestinal illnesses per 1,000 swimmers in excess of background illness rates.

The Santa Ana Region currently continues to utilize fecal coliform bacteria to assess water quality applicable to recreational beneficial uses. However, the Santa Ana Regional Board is currently conducting a triennial review of its Basin Plan, and is including an evaluation of recreational beneficial use designations and water quality objectives as part of the Basin Plan update process. We currently anticipate that the Santa Ana Regional Board will likely update fresh water bacteria water quality objectives; updated objectives may be consistent with the recommendations contained in EPA's November 2003 Implementation Guidance (see Tables 1 and 2).

ADDITIONAL GUIDELINES FOR BACTERIA

Although not enforceable as water quality objectives, Orange County beaches and bays are "posted" and access may be restricted when exceedances of certain bacteria levels are observed. The "posting" levels are described in Title 17 of the California Code of Regulations, Section 7958 (Bacteriological Standards):

The minimum protective bacteriological standards for waters adjacent to public beaches and public water-contact sports areas shall be as follows:

(1) Based on a single sample, the density of bacteria in water from each sampling station at a public beach or public water contact sports area shall not exceed:

(A) 1,000 total coliform bacteria per 100 milliliters, if the ratio of fecal/total coliform

bacteria exceeds 0.1; or
(B) 10,000 total coliform bacteria per 100 milliliters; or
(C) 400 fecal coliform bacteria per 100 milliliters; or
(D) 104 enterococcus bacteria per 100 milliliters.

(2) Based on the mean of the logarithms of the results of at least five weekly samples during any 30-day sampling period, the density of bacteria in water from any sampling station at a public beach or public water contact sports area, shall not exceed:
(A) 1,000 total coliform bacteria per 100 milliliters; or
(B) 200 fecal coliform bacteria per 100 milliliters; or
(C) 35 enterococcus bacteria per 100 milliliters.

COMPARISON LEVELS USED IN THIS REPORT

Flow Science used the following numeric values in analyzing available bacteria data:

Fecal Coliform (from existing Santa Ana Basin Plan water quality standards and Title 17 beach "posting" requirements):
- Single Sample: 400 MPN (or CFU)/100mL[5].
- Geometric Mean: 200 MPN (or CFU)/100mL.

Enterococci (from EPA-recommended criteria):
- Single Sample: 247 MPN (or CFU)/100mL.
- Geometric Mean: 54 MPN (or CFU)/100mL.

Total Coliform (from Title 17 beach "posting" requirements):
- Single Sample: 10,000 MPN (or CFU)/100mL.
- Geometric mean: 1,000 MPN (or CFU)/100mL.

Enterococci criteria used by Flow Science in this report correspond to a proposed 1.0% acceptable risk level, 95th percentile, while fecal and total coliform criteria correspond to beach posting levels. Of course, the beach "posting" requirements apply at the beach, not in upstream freshwater flows, but the numeric values provide a useful threshold value against which data can be compared.

[5] Basin Plan specifies no more than 10% of single samples to exceed this value

MONITORING DATA AND RESULTS

Flow Science examined data on bacteria concentrations from a variety of sources in the Santa Ana Region, including streams in coastal watersheds, the Santa Ana River, and inland streams. Data sources included:

- Bacteria concentrations in stream flows from Orange County coastal watersheds
- Bacteria concentrations in freshwater bodies in the Santa Ana region
- Bacteria concentration in runoff samples collected by the Los Angeles County Department of Public Works

Data from each of these sources are examined in greater detail below.

Review of Data from Orange County Coastal Watersheds

Flow Science has reviewed data from Orange County samples collected between 1986 through 2004.[6] Figures for Orange County coastal watersheds are shown in Appendix A; watersheds and data collection locations are shown in Figures A1- 2. Figures A3, A4, and A5 present long-term geometric mean concentrations, calculated as the geometric mean concentration of all available samples (including both wet and dry weather samples) for the period of record, of enterococci, fecal coliforms, and total coliforms, respectively. As shown in Figure A3, long-term geometric mean concentrations of enterococci exceed EPA's proposed freshwater enterococci water quality criteria in all the coastal creeks for which data were available. Similarly, long-term geometric mean concentrations of fecal coliform in most Newport Coast creeks exceed existing Santa Ana Basin Plan REC-1 fecal coliform water quality criteria. Figures A6, A7, and A8 present long-term geometric mean concentrations of enterococci, fecal coliform, and total coliforms plotted against the percent of development within each watershed. There is no apparent correlation for any of the three indicator bacteria presented in these figures with amount of the watershed that has been developed. Note that Figures A6 through A8 utilize the current (2005) level of development for each watershed.[7]

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[6] Data were obtained from http://www.ocbeachinfo.com/downloads/data/index.htm on February 11 and March 22, 2005. For enterococci, data were available from March 30, 1999, through December 21, 2004. For fecal coliform and total coliform, data were available from January 7, 1986, through December 21, 2004. No data were available for *E. coli*.

[7] The area of watershed that was developed was initially established by PBS&J in 1999 (PBS&J, 1999). These values have been subsequently updated based on information received from The Irvine Company in 2005. Two watersheds experienced significant development between 1999 and 2005: the Crystal Cove Creek watershed increased from ~5% to ~70% developed, and the Muddy Creek watershed increased from ~1% to ~60% developed. The level of development within the other coastal watersheds remained approximately constant.

To facilitate analysis, individual samples were segregated as follows: wet-weather[8], summer dry-weather[9], and winter dry-weather.[10] As shown in Figure A9, wet weather samples exceed single sample threshold values most frequently, regardless of which indicator bacteria are sampled (72%, 61%, and 39% of wet-weather enterococci, fecal coliform, and total coliform samples, respectively, exceed single sample thresholds). Summer dry weather samples exceed thresholds less frequently than wet-weather samples, and winter-dry weather samples exceed thresholds least frequently. The single sample thresholds used to calculate the percent of samples in exceedance are 247, 400, and 10,000 MPN/100mL for enterococci, fecal coliform, and total coliform, respectively.

Figures A10 through A53 present the following information for each site: a) a time-series scatter plot of single sample concentrations of enterococci, fecal coliform, and total coliform for the wet and dry weather data, b) wet and dry weather cumulative distribution functions for each bacteria, and c) the percentage of individual samples that exceed corresponding thresholds in each month. From this analysis, the following conclusions may be reached:

1. Lowest geometric mean concentrations of each of the three bacteria (enterococci, fecal coliform, and total coliform) occurred at the Pelican Hill Waterfall station (watershed 95% developed, primarily golf course), and highest geometric mean concentrations of each bacteria occurred at the Emerald Bay Drain station (watershed 3% developed). In the Muddy Creek watershed, which experienced substantial development between 1999 and 2005 (see footnote 7), enterococci concentrations appear to have decreased as the watershed became more developed. Trends were less evident for fecal and total coliform levels. Similar patterns emerged in data from the Crystal Cove Creek watershed, the other watershed that experienced significant development between 1999 and 2005. Enterococci and fecal coliform concentrations appear to have decreased, while any trends in the total coliform record are unclear. These results indicate that bacteria concentrations in creeks may decline as the level of development increases, and bacteria concentrations in runoff from developed watersheds may be lower than runoff from creeks in less developed coastal areas.

2. No relationship was found between the percentage of the watershed developed and the long-term geometric mean bacteria concentrations (see Figures A6, A7 and A8).

3. The time series plots indicate that concentrations of indicator bacteria are not increasing over time. By visual inspection, bacteria concentrations may be

[8] "Wet-weather" samples are those samples that were collected within two days of a rainfall event greater than or equal to 0.1 inches as measured by the Newport Beach Harbor Station.

[9] "Summer dry-weather" samples are defined as samples collected from April-November, but not within two days of rainfall greater than or equal to 0.1 inches as measured by the Newport Beach Harbor Station.

[10] "Winter dry-weather" samples are defined as samples collected from December-March, but not within two days of rainfall greater than or equal to 0.1 inches as measured by the Newport Beach Harbor Station.

decreasing over the data record in five catchments (Pelican Point Creek, Muddy Creek, Emerald Bay Drain, El Morro Creek upstream station, and Crystal Cove Creek). At the remaining six stations, no apparent long-term trend in bacteria concentration is observed. Very little if any correlation is evident between long-term trends and percentage of watershed developed, as the apparent slight decrease in bacteria concentrations was observed in watersheds that range from 1-95% developed.

4. Although Figure A9 shows that taken as a whole, wet-weather samples have higher concentrations than dry-weather samples, data from some locations show the opposite trend. At Pelican Point Creek (95% developed), dry weather concentrations for enterococci and fecal coliform are higher than wet weather concentrations. At the Emerald Bay Drain (3% developed), fecal and total coliform dry weather concentrations are significantly greater than wet weather concentrations. At El Morro Creek (1% developed), Broadway Creek (25% developed), and Crystal Cove Creek upstream station (70% developed) there is no significant difference (by visual inspection of Figures A34-36, A50-52, and A38-40, respectively) between wet and dry weather bacteria concentration distributions.

5. The general observation that winter dry-weather samples on average contain fewer bacteria than summer dry-weather samples is evident in many of the scatter plots. Figures A10, A34, A38, A42, and A46 (presenting data from Pelican Point Creek, El Morro Creek, Crystal Cove Creek upstream, Crystal Cove Creek, and Buck Gully) illustrate this behavior most clearly.

These results are consistent with the results from an earlier study (PBS&J, 1999) in which long-term geometric mean concentrations of bacteriological data from November 1996-October 1999 were evaluated.

Bacteria Concentrations in Inland Waters in the Santa Ana Region

As part of the activities conducted by the Stormwater Quality Standards Task Force, CDM has compiled bacteriological data from several agencies within the Santa Ana Region (CDM, 2005). The CDM study included data collected and compiled by Orange County, the Regional Water Quality Control Board (Region 8), the Santa Ana Watershed Project Authority, the County of San Bernardino, the County of Riverside, the United States Environmental Protection Agency (EPA), the United States Geological Survey, and Orange County Coastkeeper. Select figures produced by CDM in this study are shown in Appendix B. CDM performed an overview analysis of all bacteria data collected, and reached the following broad-based and general conclusions:

1. Concentrations of indicator bacteria in samples collected from inland water bodies very frequently exceed existing Basin Plan fecal coliform water quality objectives and EPA-proposed *E. coli* criteria.

2. Bacteria concentrations in samples obtained from upstream, largely undisturbed areas are typically lower than those in samples from downstream areas affected by urbanized land uses. Concentrations in upstream samples are more frequently below water quality objectives and proposed criteria than downstream samples.

3. Winter dry-weather samples are more likely to meet objectives than summer dry-weather samples, consistent with results from the Orange County coastal watersheds.

CDM also conducted a detailed analysis of six sites[11] for which long-term data records were available. These six sites exhibited varying degrees of urbanization and channel modification. A map showing the locations of these six sites is shown in Appendix B as Figure B1. Detailed results from these stations are reproduced in Appendix B as Figures B2 through B13. Land use distributions for the areas tributary to the study sites are shown in Table 3.

Table 3. Approximate land use distributions in the watersheds of CDM's six detailed study sites

Site	% Vacant	% Residential	% Commercial	% Industrial	% Other
Chino Cr.[a]	3.2	61.3	16.7	9.7	9.1
Santa Ana Delhi Channel	0.9	52.4	26.0	9.2	11.5
Temescal Cr.	67.3	16.2	2.4	3.4	10.7
Santa Ana R. at Imperial Highway[b]	-	-	-	-	-
Santa Ana R. at MWD Crossing[c]	-	-	-	-	-
Icehouse Canyon Creek	100	0	0	0	0

a) Chino Creek land use data are for portion of watershed downstream of San Antonio Dam.
b) CDM concluded that any potential relationship between land use and bacteria concentrations in this reach of the Santa Ana River is likely masked by the interception of flows by Prado Dam; consequently, no data land use data were available in the CDM report for this site.
c) CDM did not include land use statistics for this station in its report. The report states that land use is "diverse…a combination of commercial, residential, industrial, and agricultural lands. The upper part of the watershed includes natural undeveloped lands…Residential land is dispersed throughout the contributing area."

11 The six sites examined by CDM include: Chino Creek at Schaeffer Avenue, the Santa Ana Delhi Channel, Temescal Creek at Lincoln Avenue, the Santa Ana River at Imperial Highway, the Santa Ana River at the Metropolitan Water District crossing, and Icehouse Canyon Creek in the Angeles National Forest.

By examining these sites in detail, CDM found the following:

1. In streams where flow rate data are available, high bacteria counts are in many cases but not always associated with high flow events (presumably caused by rainfall). Bacteria concentrations in samples collected from Chino Creek at Schaeffer Avenue (Figure B2) and the Santa Ana Delhi Channel (Figure B3) are frequently elevated and do not exhibit any apparent correlation with flow rate in the channel. In Temescal Creek (Figure B4) and the Santa Ana River at the MWD crossing (Figure B5), the data are widely scattered and patterns are difficult to detect. In the Santa Ana River at Imperial Highway (Figures B6-7), data show that bacteria levels are elevated during high flow events and the levels remain elevated for 1-2 days after the high flow has receded.

2. Bacteria concentrations appear to be decreasing over time at three locations (Chino Creek at Schaeffer Ave. (data record 2002-2004), Santa Ana River at MWD Crossing (data record 1984-2004), and Santa Ana River at Imperial Highway (data record 1981-2004)). At the other three locations, no long-term trends are apparent.

3. All sites except Icehouse Canyon Creek have regularly exceeded current or proposed water quality objectives. As mentioned previously, concentrations at the two Santa Ana River sites have shown a decreasing trend, and since 1998 most samples have been at or below objective levels. Icehouse Canyon Creek, at elevation 5,100 feet in the Angeles National Forest, has only one sample (of 40 total samples; a fecal coliform measurement of 9,400 MPN/100mL) in the data record that does not comply with existing or anticipated water quality objectives, indicating that runoff from remote, undeveloped, forested catchments at higher elevations may have significantly lower bacteria levels than runoff from lower elevation watersheds, including undeveloped watersheds at lower elevations. Figures B8-13 show, for each of the six sites, the percent of months in which single sample thresholds are exceeded when samples are classified as summer dry, winter dry, or wet-weather.

Los Angeles County Monitoring Data

Los Angeles County has prepared an Integrated Receiving Water Impacts Report (Los Angeles County, 2001), which includes bacteria concentrations measured in runoff collected downstream of catchments that exhibited primarily single land use types. Los Angeles County data for indicator bacteria for several major land use types are shown in Table 4 (adapted from Table 4-12 of the L.A. County report).

Table 4. Bacteria concentration means, medians and coefficients of variation (C.V.) from Los Angeles County Land Use Sites

	Total Coliform			Fecal Coliform			Enterococcus		
Land Use Type	**Mean**	**Median**	**CV[a]**	**Mean**	**Median**	**CV[a]**	**Mean**	**Median**	**CV[a]**
Commercial	1,140,000	1,250,000	0.71	528,750	90,000	1.35	86,250	40,000	1.18
Vacant	9,187	2,200	1.25	1,397	500	2.60	679	500	0.98
High density S.F. residential	1,366,667	1,600,000	0.30	933,333	900,000	0.70	610,000	140,000	1.41
Transportation	692,500	600,000	0.82	328,750	205,000	1.22	32,000	32,000	0.65
Light industry	454,000	160,000	1.42	338,220	30,000	2.09	98,200	130,000	0.73

a) "CV" refers to "Coefficient of Variation", calculated by dividing the standard deviation by the mean.

The data shown in Table 4 demonstrate that significantly lower bacteria concentrations were observed in runoff from vacant land areas than in other land use types. These data were collected by Los Angeles County in Sawpit Creek, downstream of Monrovia Creek, in the City of Monrovia; this catchment is in the San Gabriel Mountains in a very steep, sparsely vegetated area far from the ocean. Low concentrations of indicator bacteria from the Sawpit Creek watershed are consistent with low concentrations in samples collected from Icehouse Canyon Creek, both mountainous, high elevation watersheds. These results differ from observations from the Orange County coastal watersheds, which indicate no relationship between percentage development in a watershed and bacteria concentrations. The differences are most likely due to differences in catchment characteristics, local climate, the numbers and types of wildlife present, or to other factors. In any case, both the mean and median concentrations observed for each Los Angeles County land use type exceeded applicable water quality thresholds.

Los Angeles County also measured bacteria concentrations in several "mass emission" stations. These stations were sited to capture runoff from major Los Angeles County watersheds that generally have heterogeneous land use, with the objective of estimating pollutant loads to the ocean and of identifying long-term trends in pollutant concentrations, where possible. The mass emission stations include Malibu Creek (watershed 6% impervious; measurement station near Malibu Canyon Road), Ballona Creek (watershed 45% impervious; measurement station between Sawtelle Boulevard and Sepulveda Boulevard in Los Angeles), the Los Angeles River (watershed 35% impervious; measurement station between Willow Street and Wardlow Road in Long Beach), and the San Gabriel River (watershed 30% impervious; measurement station below the San Gabriel River Parkway in Pico Rivera).

In addition to the land use data reported in Table 4, Los Angeles County reached a number of conclusions using data collected at these mass emission stations. The following conclusions are cited directly from the Los Angeles County report (2001):

- The Malibu Creek station appears to have consistently lower [bacteria] counts than other mass emission stations.

- Every wet weather mass emission bacteria sample taken exceeded the public health criteria for indicator bacteria. All of the dry weather bacteria samples taken for the low flow diversion projects exceeded the public health criteria. Most of the dry weather mass emission bacteria samples taken exceeded the public health criteria. Wet weather flows contained bacteria densities at much higher levels (three to four orders of magnitude) than dry weather flows.

- Except for 1996-97, densities observed during the first storm of each rainy season were not necessarily higher than during consecutive storm events, suggesting that there was no consistent "first-flush" effect in these watersheds. Peak densities were observed at different times each year. In 1995-96, the peak density at all four mass emission stations and one land use station coincided with the peak storm of the season.

- Except for somewhat lower [bacteria] densities at Malibu Creek, there was no seasonal or regional consistency in cell densities. There was a very wide range of densities for all stations.

Consistent with data from Orange County coastal watersheds, the Los Angeles County data show that samples collected during wet-weather exhibit significantly higher bacteria concentrations than samples collected during dry weather.

ADDITIONAL DATA ON SOURCES AND CONCENTRATIONS OF BACTERIA IN RUNOFF

Numerous additional studies and data reports have shown a correlation between elevated bacteria concentrations and rainfall events in Southern California. This correlation is evident in data collected from a variety of environments. For example, elevated concentrations of indicator bacteria have been observed during wet weather conditions at Huntington Beach (Boehm et al.,2002; Kim et al., 2004; Reeves et al., 2004), and northern Orange County and Santa Cruz County (Dwight et al., 2004).

Several studies also indicate that runoff from undeveloped watersheds contains bacteria concentrations that exceed relevant water quality standards. For example, storm water runoff from the head of the Rose Creek watershed in the San Diego Region contains levels of indicator bacteria well in excess of water quality objectives, even though this area is non-urban, contains no sewer lines or lift stations, and is restricted from public access (Schiff and Kinney, 2001). Moore (2001) found that concentrations of indicator bacteria in San Juan Creek sampling stations reflecting rural land uses exceeded water quality criteria, and that rainfall events resulted in higher bacteria concentrations at both rural and urban sites than dry weather. (Moore (2001) also found that storm drains can be major sources of dry weather bacteria pollution.)

The level or type of development is not necessarily indicative of bacteria levels in runoff, or

of the presence of human-derived bacteria. In Mission Bay, a highly urbanized watershed, extensive efforts have been made to eliminate human sources of bacteria by repairing the sanitary sewer system and diverting dry weather flows to a local waste water treatment plant. Source tracking studies suggest that human sources contribute a minor fraction of the total fecal inputs to the Bay, and yet violations of water quality standards continue to occur (see Colford et al., 2005, and references therein). Pednekar et al. (2005) also found that changes in land use associated with the development of agricultural lands[12] within watersheds tributary to Newport Bay did not have a significant impact on bacteria loads, stating "The storm loading rate of coliform…appears to be unaffected by the dramatic shift away from agricultural land-use."

A number of studies have indicated that runoff from urban areas may not be the sole or even the primary source of elevated bacteria concentrations in receiving waters, but that such elevated levels may be caused by non-human sources, such as terrestrial wildlife and birds or even local sediments. Studies conducted at Huntington Beach have indicated that there may be many sources of indicator bacteria to the surf zone, including urban runoff, flow from adjacent wetlands, birds, and sediments (Grant et al., 2001). A recent study by Noblet et al. (2004) indicates that birds may be the source of high concentrations of indicator bacteria at the mouth of the Santa Ana River and in the nearby surf zone, and suggested that local sediments may be the source of fecal steroids, indicating the presence of fecal-associated material in the sediments. Another study by the Los Angeles Regional Water Quality Control Board (2004) erected a bird exclusion structure on Cabrillo Beach, and found that bacteria levels below the structure were reduced up to 60% compared to levels measured outside the structure, while exceedances of water quality standards were reduced by 65% below the structure. The Los Angeles Regional Board also reported that "high bacterial densities may be largely from the beach itself."

Other studies have provided additional evidence that the bacteria found in creeks may result from natural, not urban, sources. Orange County recently studied the efficacy of several best management practices (BMPs) for reducing bacteria concentrations in Aliso Creek, Orange County, California. Results of this study have been summarized by GeoSyntec (2005) (attached as Appendix C). The BMPs that were evaluated include 1) a multimedia filtration and UV sterilization system, and 2) wetland ponds. The study, which was conducted during dry weather, found that both BMPs greatly reduced concentrations of indicator bacteria[13], but that bacteria levels rebounded within a short distance downstream of the BMPs. In the case of the filtration/sterilization, the geometric mean concentration of fecal coliform increased from 317 cfu/100mL at the outlet of the BMP to

[12] Tributary creeks to Newport Bay studied by Pednekar et al. include the San Diego Creek (SDC) and the Santa Ana Delhi Channel (SAD). The SDC watershed remained between 52-60% developed over the study period. Agricultural land-use decreased from 34% to 2%, while commercial land-use increased from 1% to 10%, industrial land-use from 2% to 20%, and residential land-use from 11% to 25%. The SAD watershed remained between 88-92% developed over the study period. Agricultural and residential land-use decreased while commercial land-use increased from 3% to 15% and industrial land-use increased from 19% to 33%.

[13] In comparing influent and effluent, multimedia filtration/UV sterilization resulted in a 99.6% reduction in fecal coliform concentration; wetland ponds achieved a 90-99% reduction in fecal coliform concentrations.

2575 cfu/100mL in a natural channel at a distance of 35 feet downstream of the BMP. In the case of the wetland ponds, effluent was routed through a pipe approximately 200 feet long to the monitoring station, which recorded concentrations approximately two times greater than what could be accounted for based on mass-balance calculations. However, uncertainty in flow measurements, data variability, and the fact that ~37% of the flow is not intercepted by the wetlands indicate that regrowth is not the only possible explanation for the unexpectedly high bacteria concentrations at the pipe outlet.

The link between bacteria concentrations in rivers and streams and downstream water quality, including surf zone water quality, has been examined by a number of authors in addition to those cited above. PBS&J (1999) found that even though Newport coastal creek waters contained high concentrations of indicator bacteria, it did not appear that these waters had a significant impact on bacteria concentrations in the surf zone. Ahn et al. (2005) found that while storm water runoff from the Santa Ana River may lead to "very poor" surf zone water quality, the impact on the surf zone was generally confined to <5 km around the river outlet. Pednekar et al. (2005) studied bacteria concentrations in Newport Bay, California, and found that approximately 70% of the variability in the coliform record could be attributed to rainfall, implying that storm water runoff from the surrounding watershed is a primary source of coliform in Newport Bay. A difference in scale may account for the different conclusions reached by different studies – the Ahn et al. and Pednekar et al. studies found significant impacts on surf zone water quality by examining large creeks and rivers, while PBS&J's conclusion that creek water quality does not significantly affect surf zone water quality is based on a study of small to medium sized creeks – and clearly highlights the need for site-specific evaluations of bacterial water quality.

Presumably, the source of bacteria affects its pathogenicity and risk to human health, but data on human health risks from non-human source bacteria are scarce. Some studies (see, e.g., Schroeder et al., 2002) call into question whether the presence or concentration of indicator bacteria in urban runoff has any relationship with the possible presence of human pathogens. Schroeder et al. sampled paved and grass areas of parks, roofs, residential lawns, ponds, storm drains and similar surfaces to characterize the microbial community that may be present in urban water. Each sample was tested for indicator organisms (coliforms, fecal coliforms, *E. coli*, and enterococci), viruses (adenovirus, enterovirus, hepatitis A virus, and rotavirus), bacteria (enterohemorrhagic Escherichia coli, enterotoxigenic Escherichia coli, Shigella, Salmonella, and Staphylococcus aureus), and protozoa (Giardia lamblia and Cryptosporidium parvum). The study states found that although pathogens can be found in urban drainage, "there does not appear to be a relationship between the presence of pathogens and the concentration or presence of indicator organisms." Of particular note, a recent epidemiological study of health risks due to swimming in Mission Bay (Colford et al., 2005), where concentrations of indicator bacteria are believed to be predominantly from non-human sources, concluded that the risks of swimming-related illness were uncorrelated with exceedances of state water quality thresholds or with levels of indicator bacteria.

In conclusion, the available data from Southern California indicate that bacteria concentrations are often elevated in runoff from both urban and undeveloped watersheds,

particularly during wet weather conditions. The level of development appears to have little effect on bacteria concentrations in storm flows. There is no clear trend in bacteria concentrations over time, with concentrations remaining relatively steady, even in areas where land use characteristics have changed over time. Available data also indicate that multiple sources may contribute to high concentrations of indicator bacteria, including natural sources such as wildlife, birds, and sediments. Regrowth within the environment also occurs, resulting in elevated bacteria concentrations even downstream of the point where relatively bacteria-free flows enter natural channels or man-made conveyances. Finally, the impact of high bacteria concentrations on downstream water quality appears to vary by location and conditions.

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APPENDIX A

DATA FROM ORANGE COUNTY COASTAL CREEKS

Figure A 1: Location of coastal catchments and surf zone areas along the Newport Coast.

Figure A 2: Additional detail on the catchment areas (information collated from the PBS&J report, 1999 and updated by The Irvine Company, 2005).

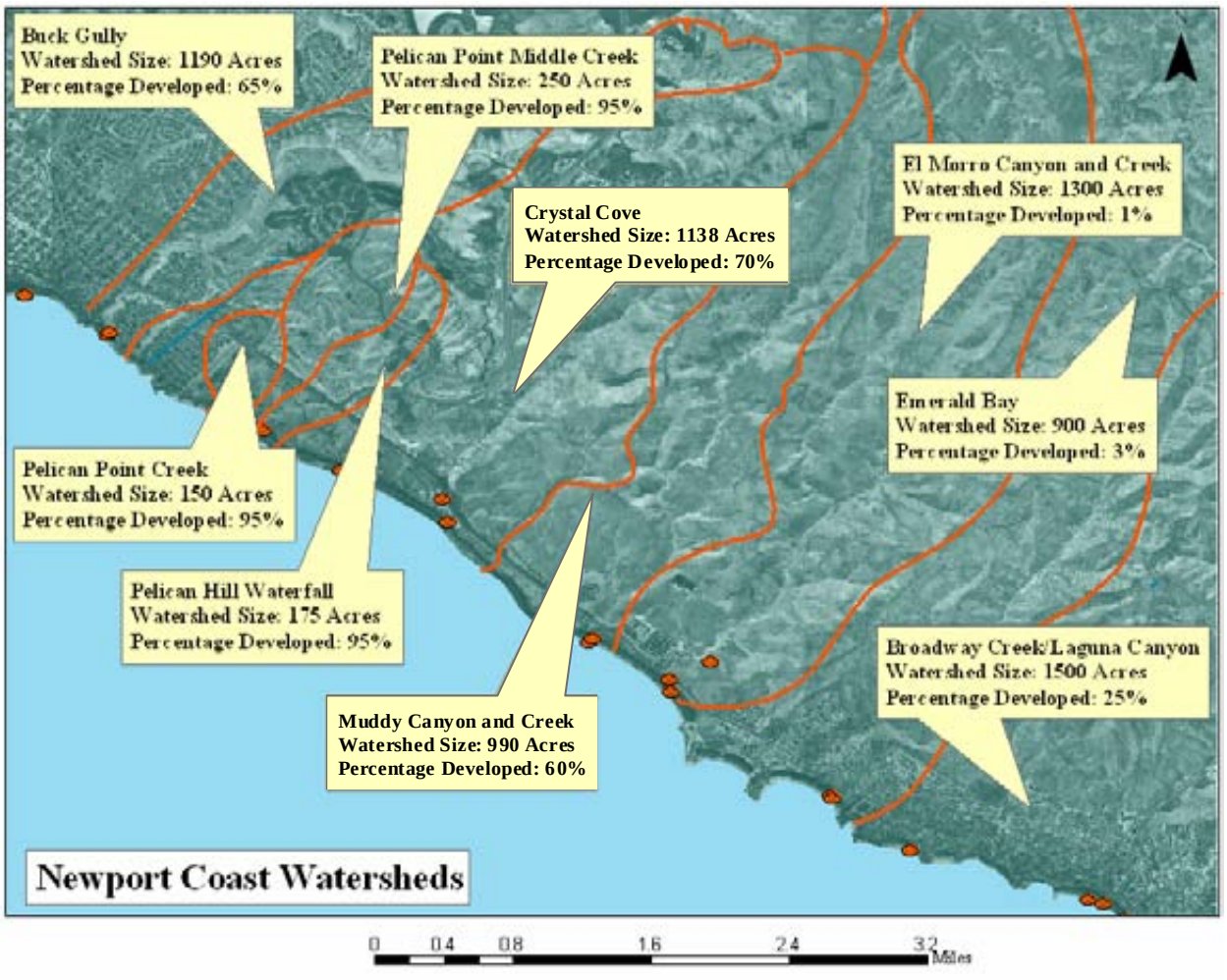


Figure A 3: Long-term geometric mean concentration for enterococci (data from 3/30/99 to 12/21/04). Dashed line represents EPA's suggested 30-day geometric mean water quality criterion for enterococci corresponding to a 1.0% risk level.

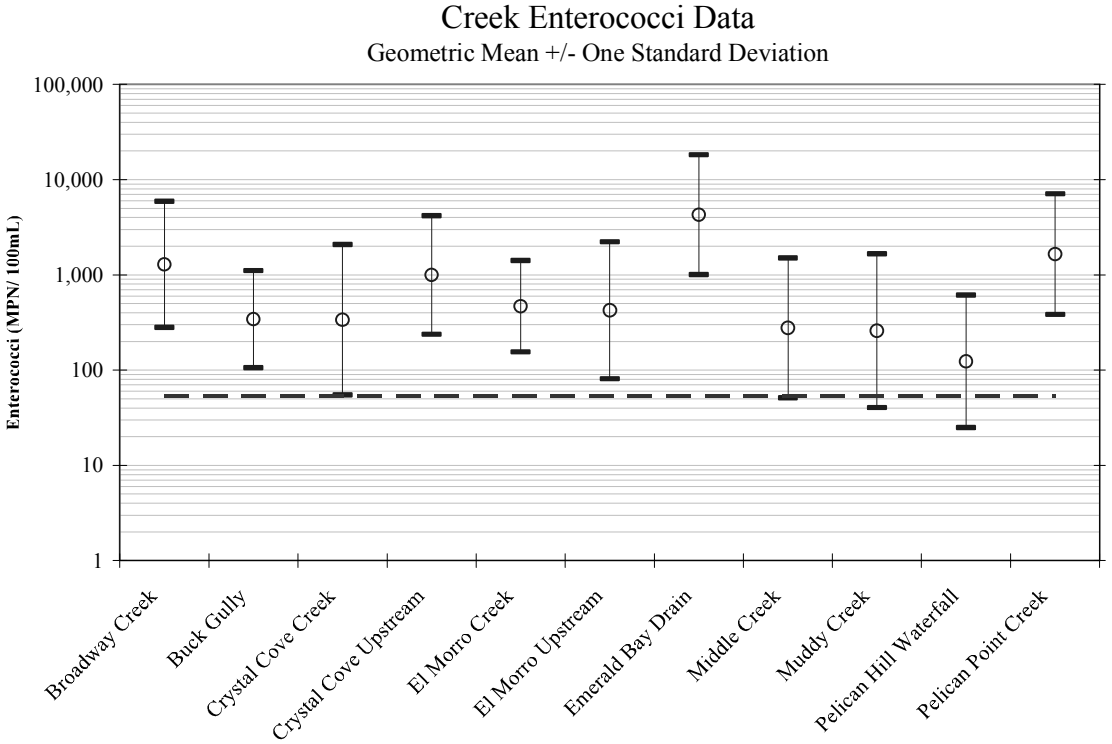


Figure A 4: Long-term geometric mean fecal coliform concentrations (data from 1/7/86 to 12/21/04). Dashed line corresponds to the current Santa Ana Basin Plan water quality criterion for 30-day log mean (geometric mean) fecal coliform concentrations.

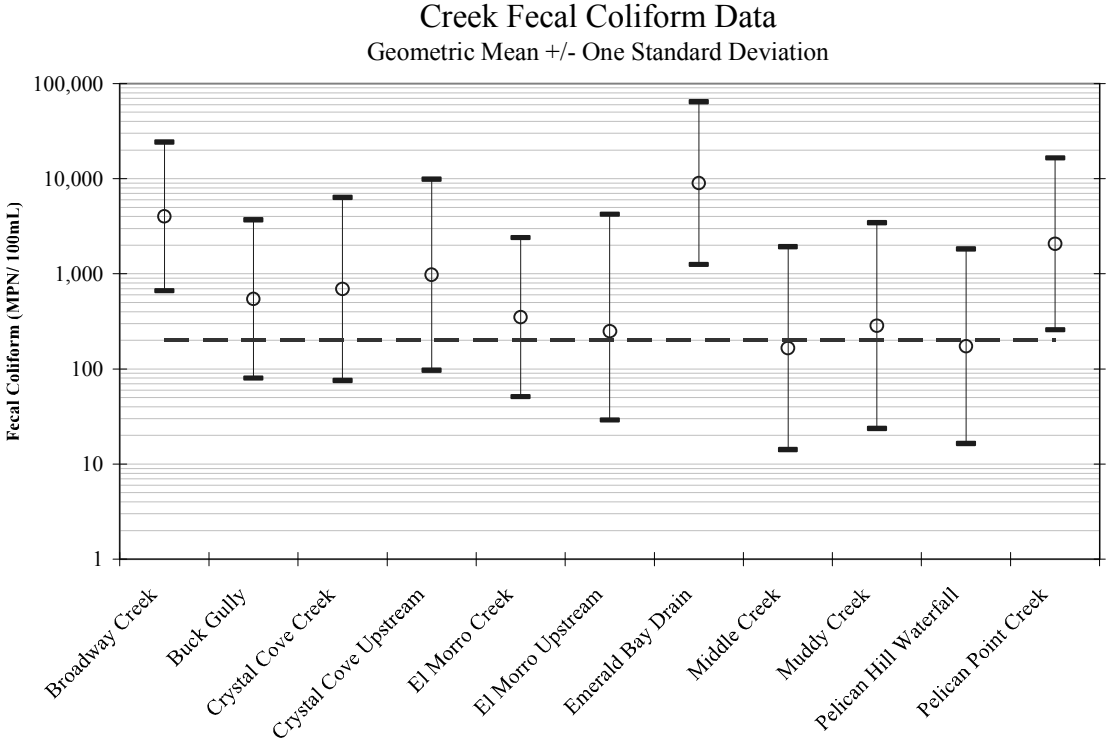


Figure A 5: Long-term geometric mean concentrations for total coliform (data from 1/7/86 to 12/21/04)

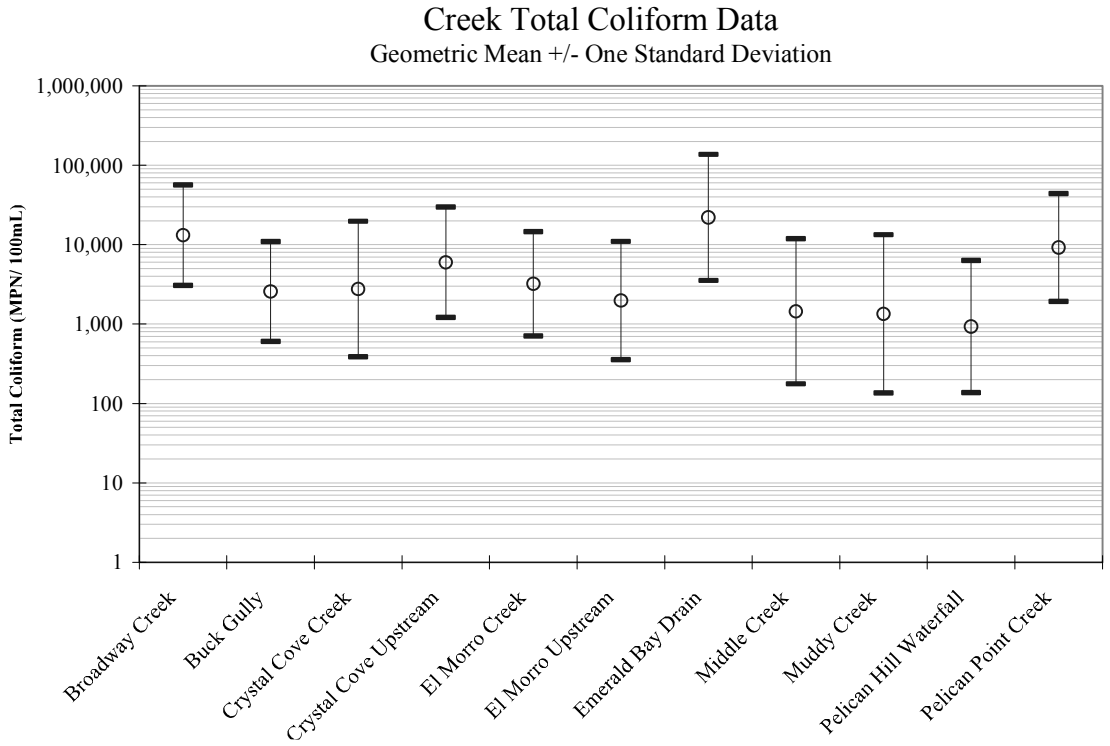


Figure A 6: Relationship between % developed and the long-term geometric mean enterococci concentration (data from 3/30/99 to 12/21/04). Dashed line represents EPA's suggested 30-day geometric mean water quality criterion for enterococci corresponding to a 1.0% risk level.

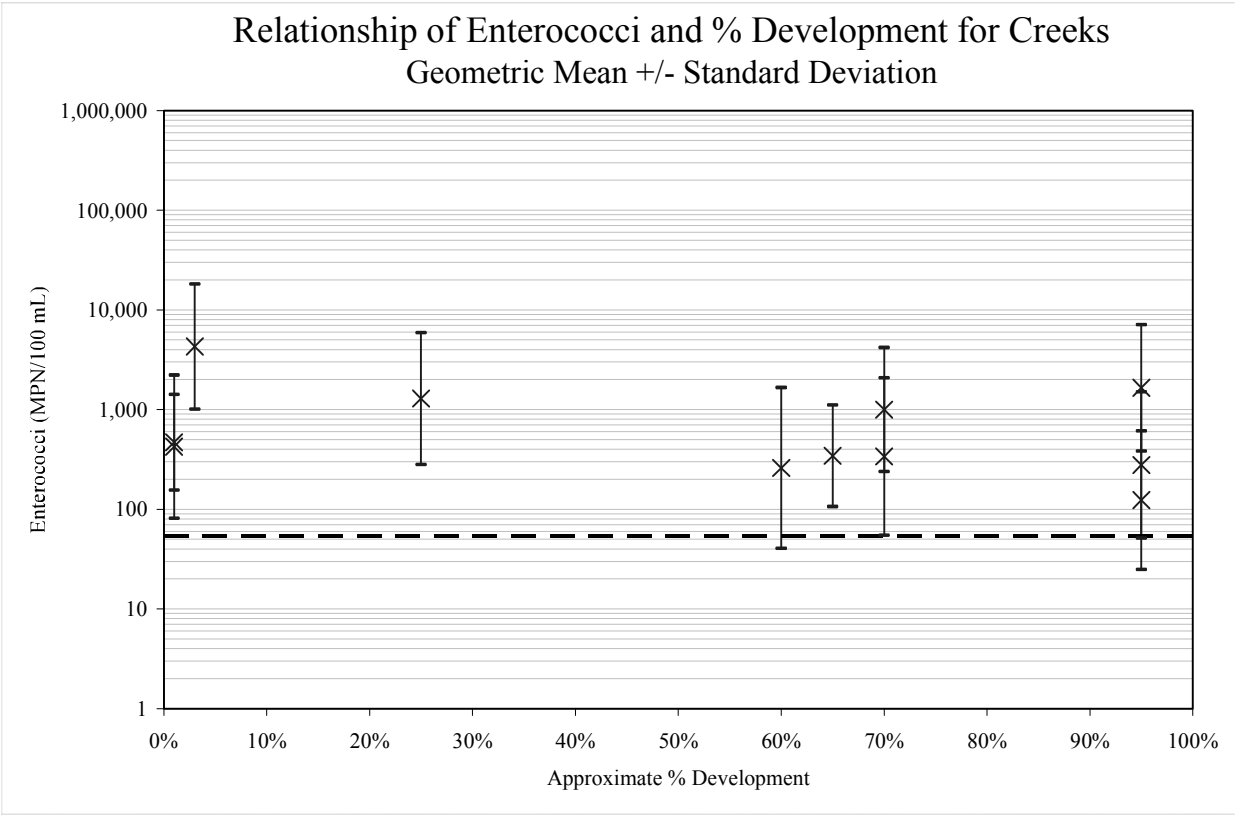


Figure A 7: Relationship between % developed and the long-term geometric mean fecal coliform concentration (data from 1/7/86 to 12/21/04). Dashed line corresponds to the current Santa Ana Basin Plan water quality criterion for 30-day log mean (geometric mean) fecal coliform concentrations.

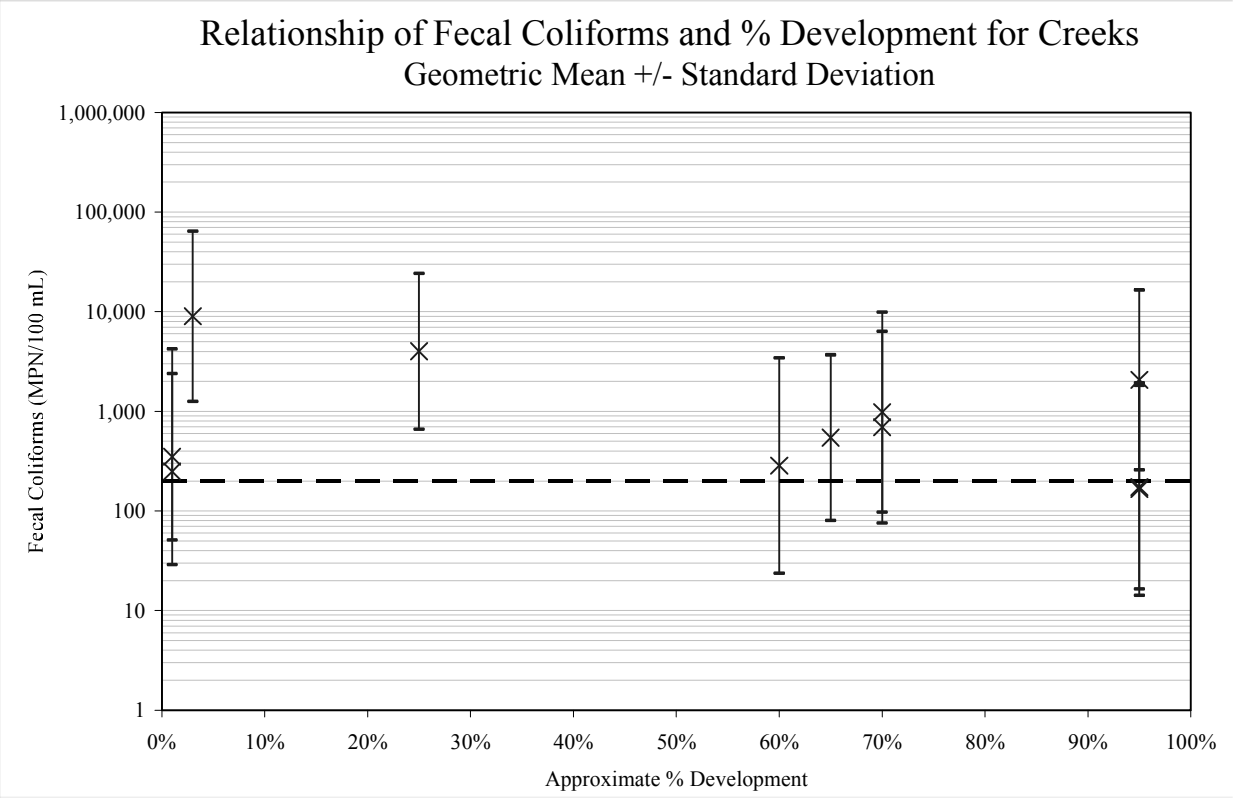


Figure A 8: Relationship between % developed and the long-term geometric mean total coliform concentration (data from 1/7/86 to 12/21/04).

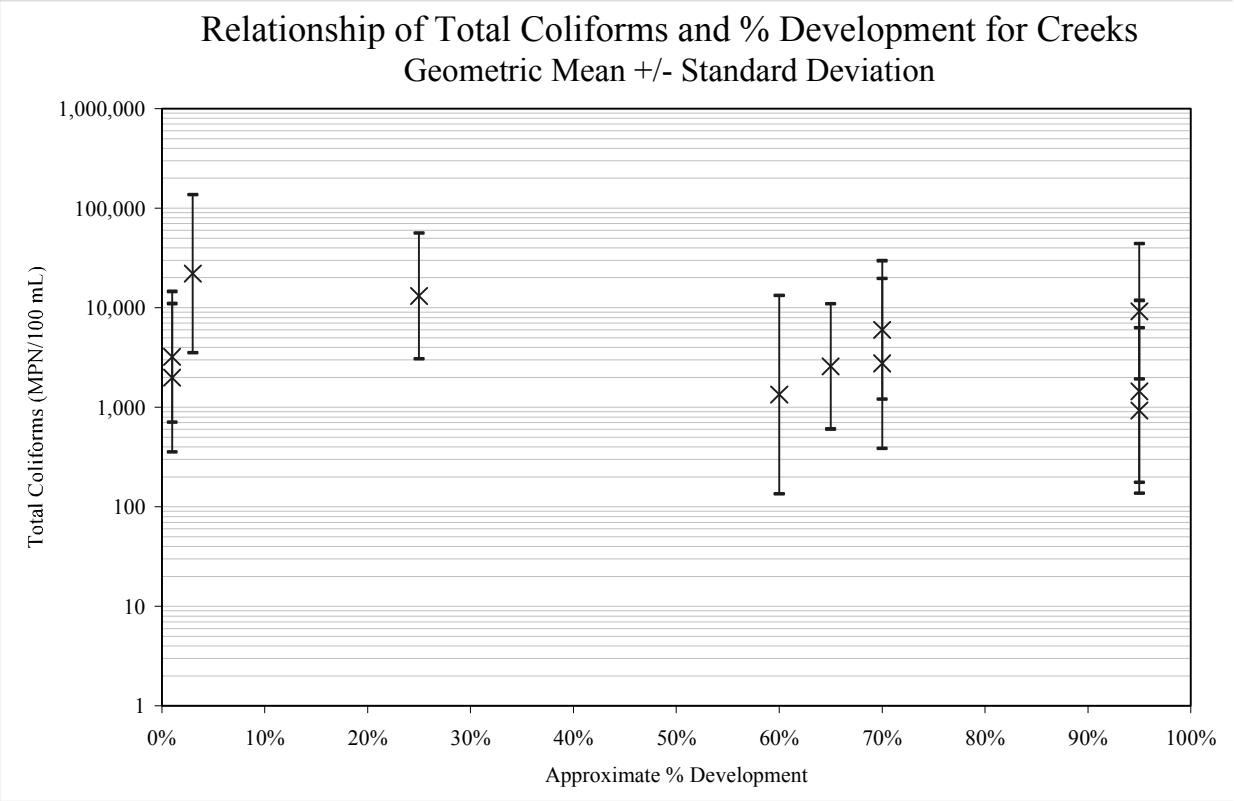


Figure A 9: Percent of samples in exceedance of thresholds by weather type (data from 1/7/86 to 12/21/04 for total and fecal coliform and from 3/30/1999 to 12/21/04 for enterococci). "Wet" data are those within two days of rainfall totaling 0.1" or greater at Newport Harbor. "Summer Dry" samples were collected from April-November, but not within two days of 0.1" or more of rain. "Winter Dry" samples were collected from December-March, but not within two days of 0.1" or more of rain. Threshold values against which data were compared are 247, 400, and 10,000 MPN/100mL, for enterococci, fecal coliform, and total coliform, respectively.

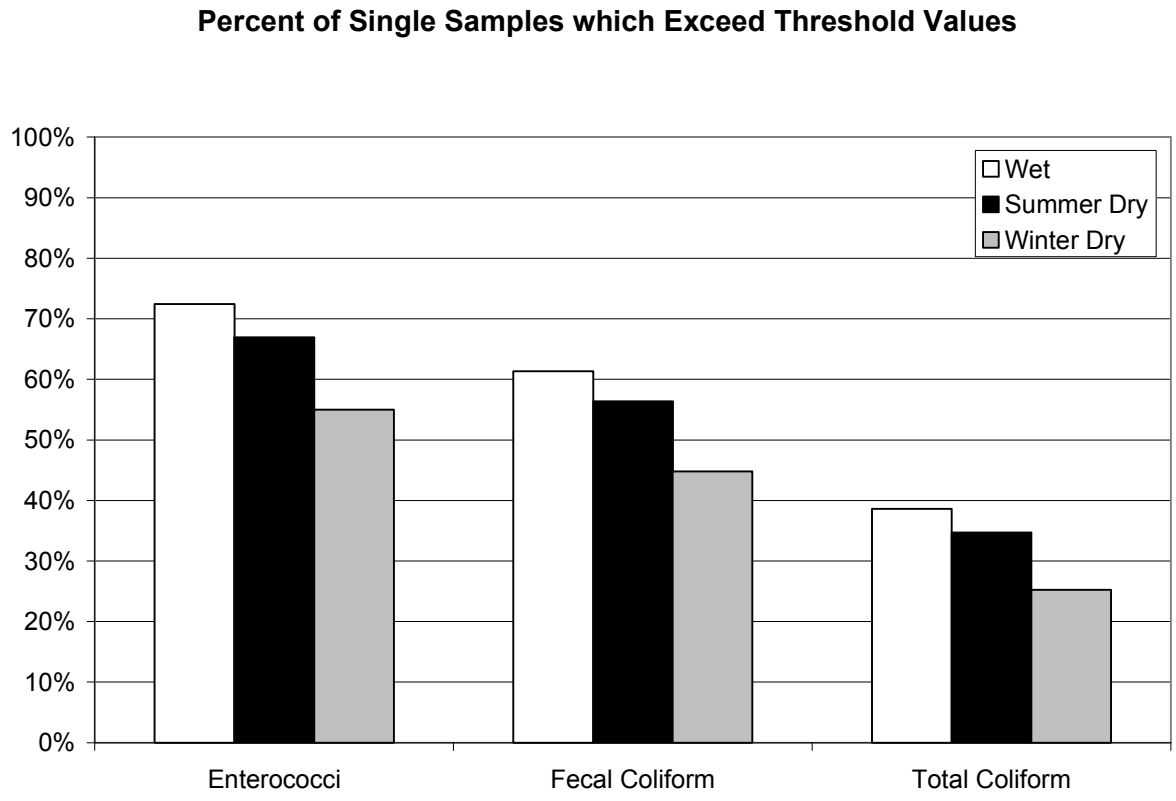
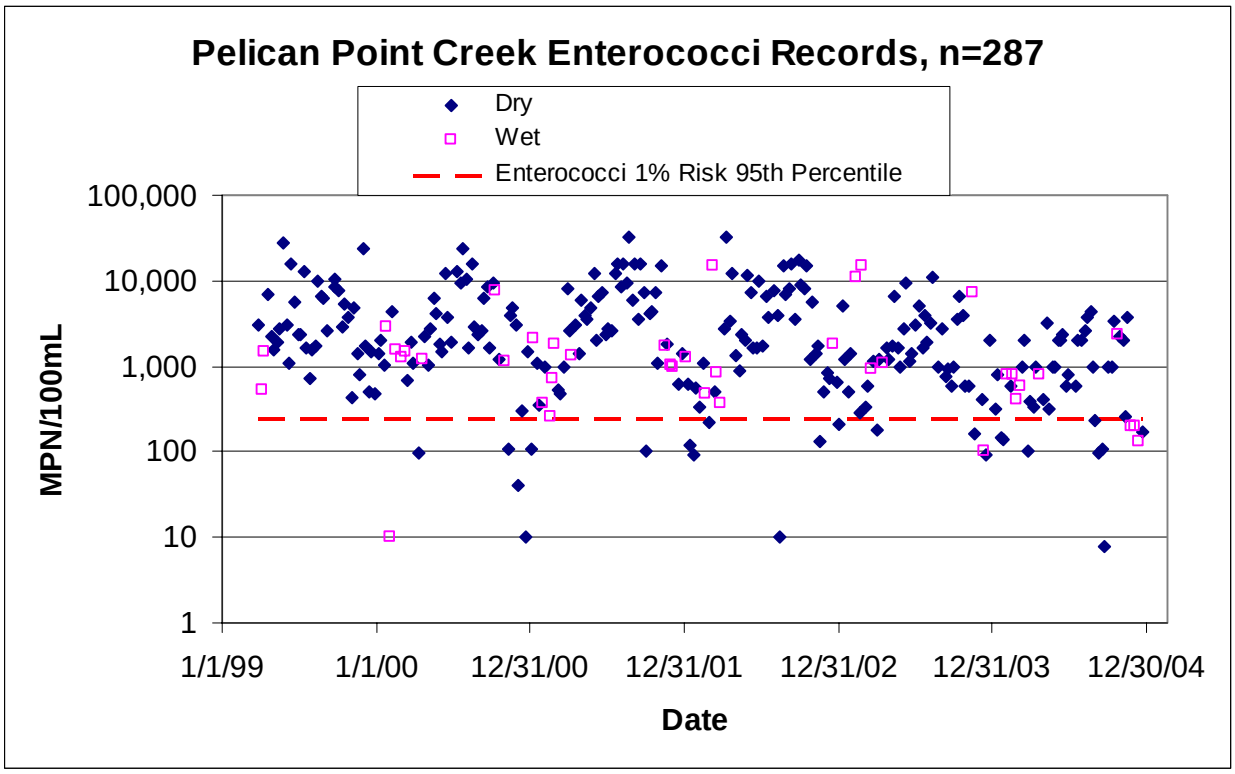


Figure A 10: Pelican Point Creek enterococci data and corresponding cumulative frequency distribution



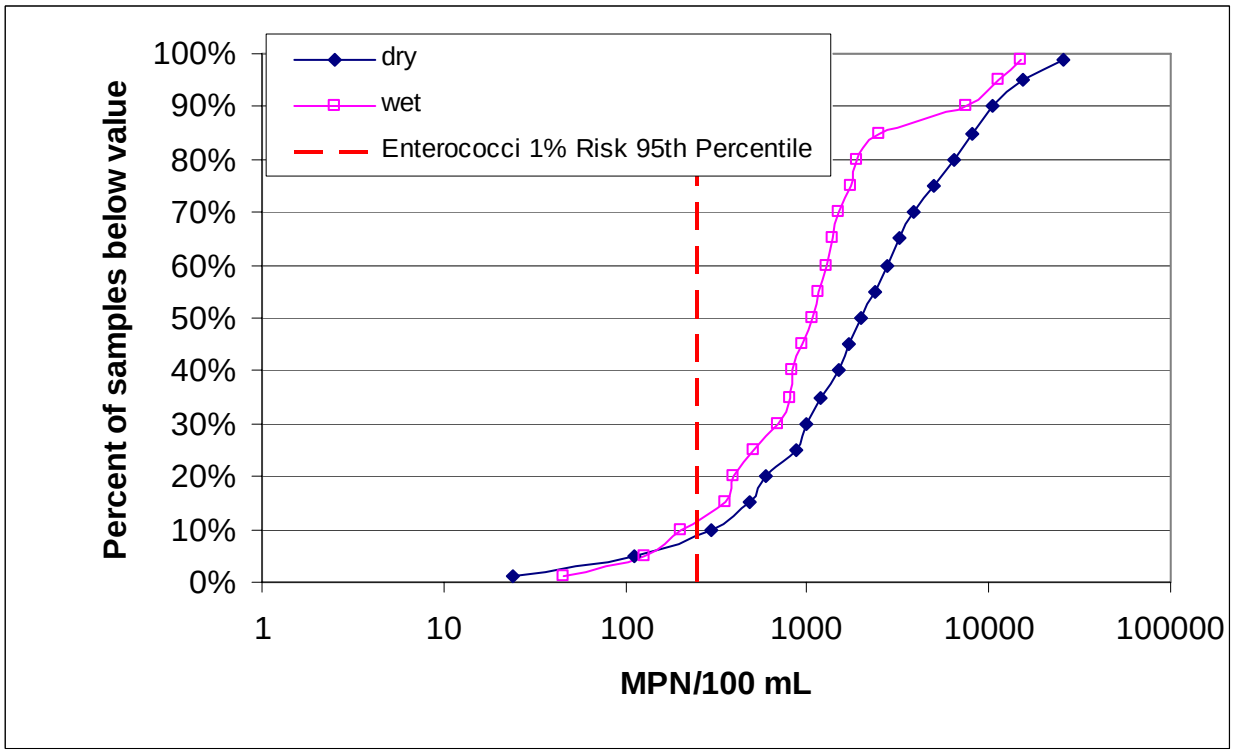
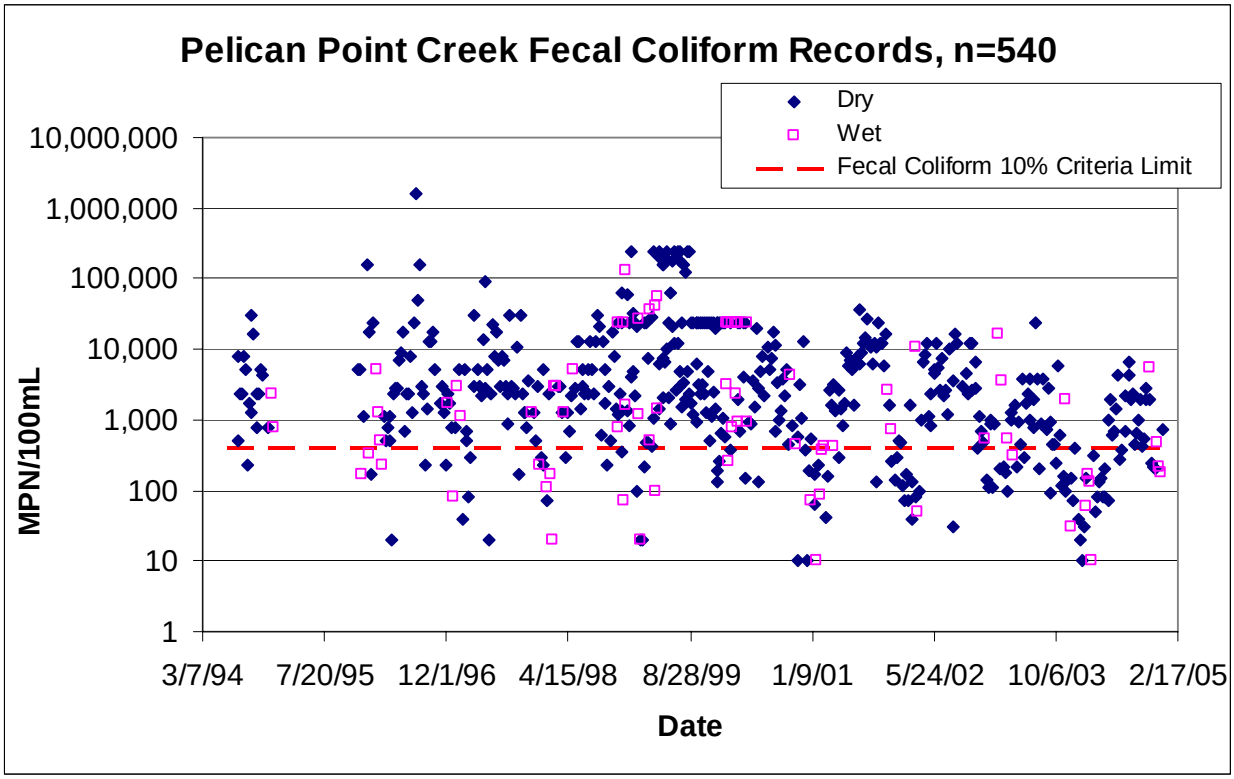


Figure A 11: Pelican Point Creek fecal coliform data and corresponding cumulative frequency distribution



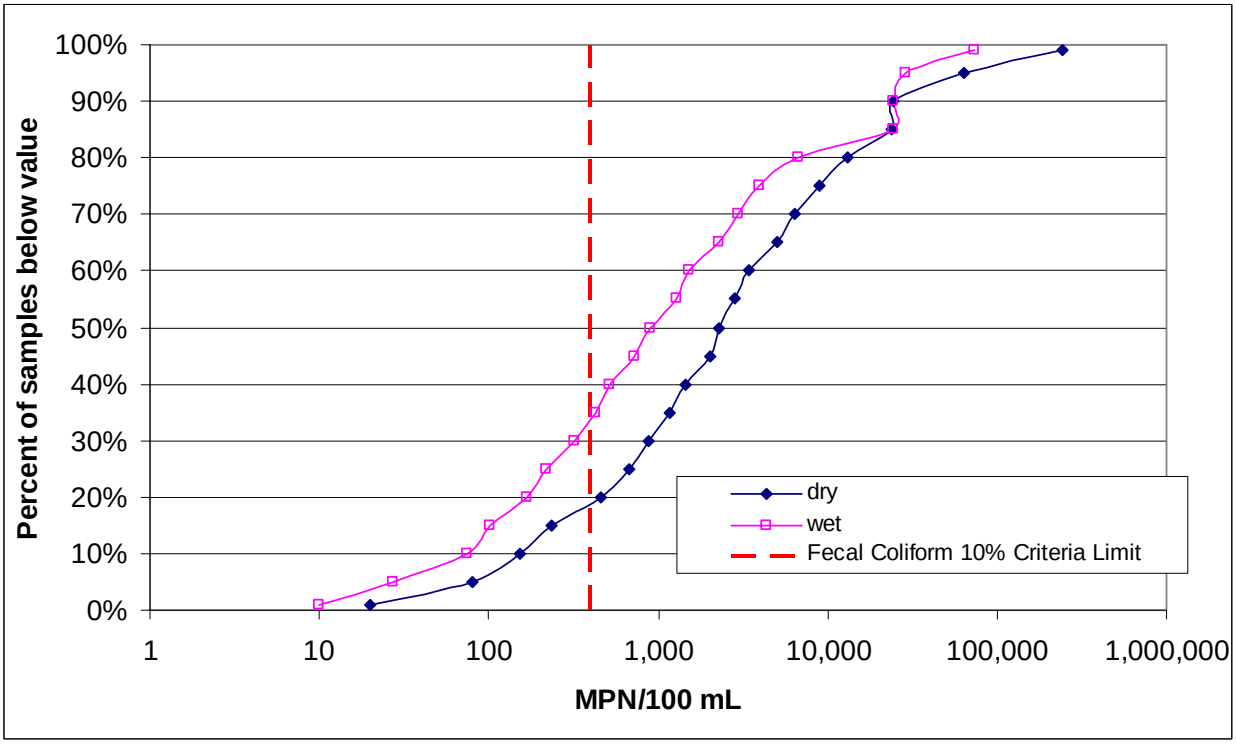
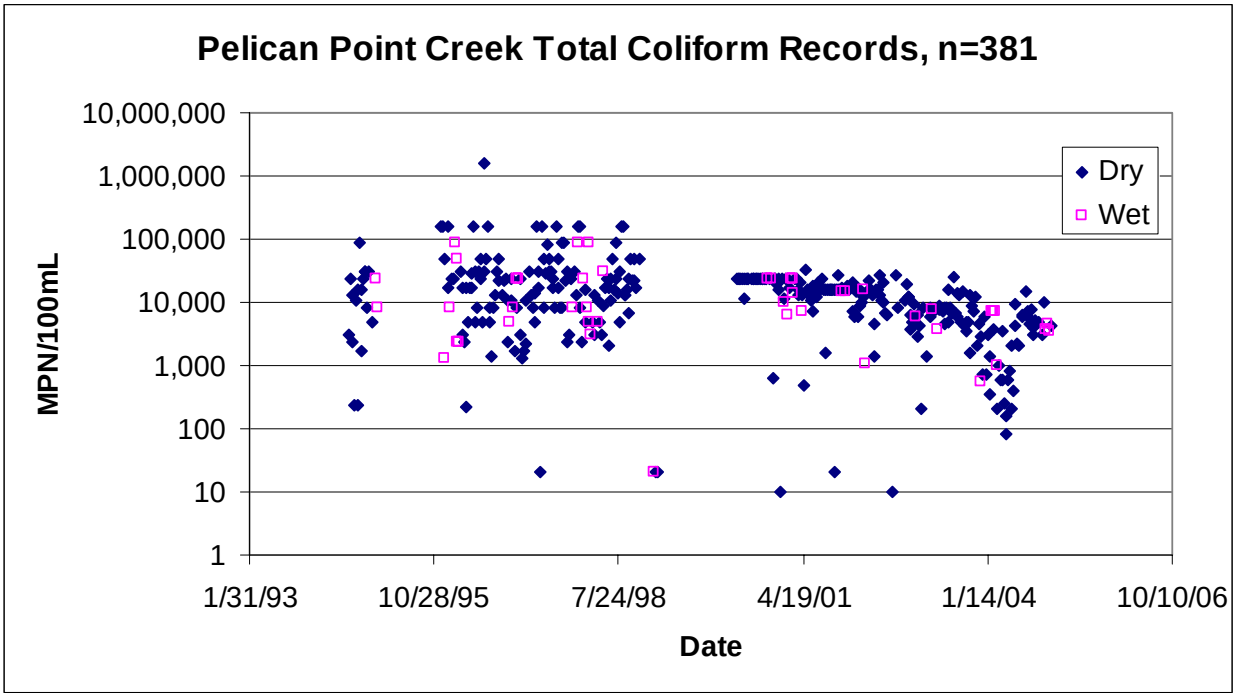


Figure A 12: Pelican Point Creek total coliform data and corresponding cumulative frequency distribution



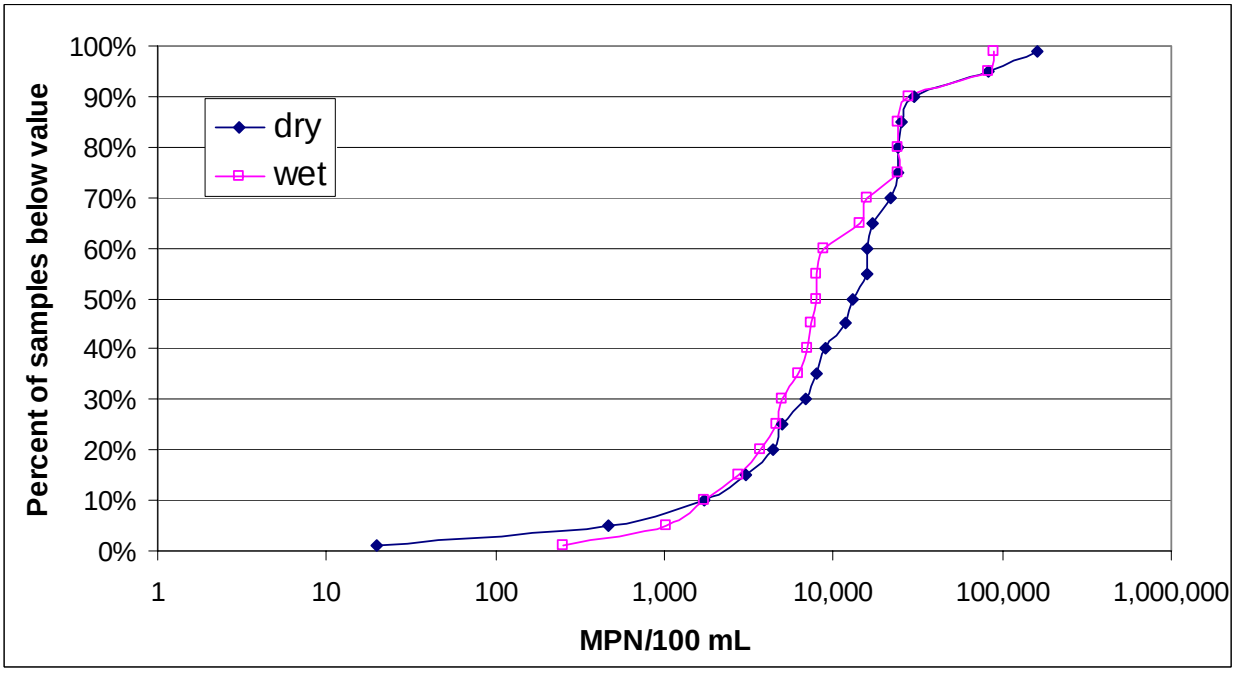


Figure A 13: Percentage of samples from Pelican Point Creek which exceed thresholds, by month

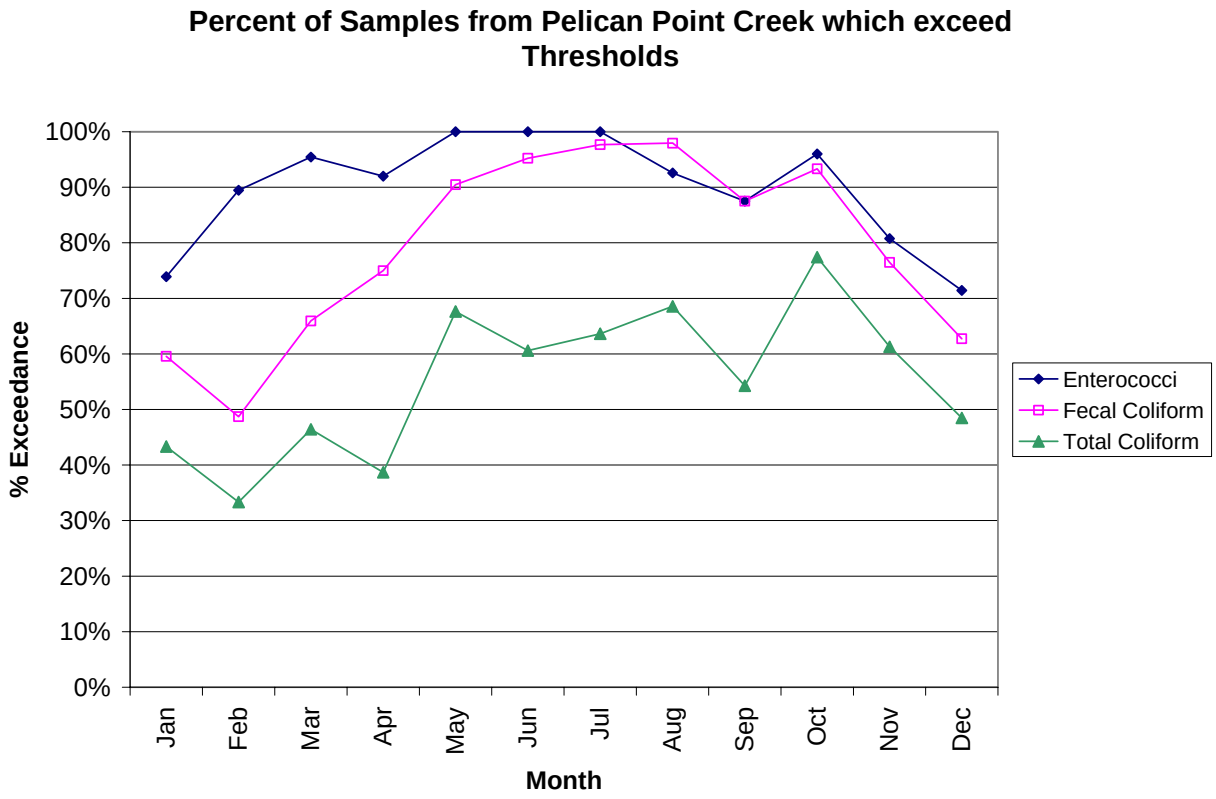
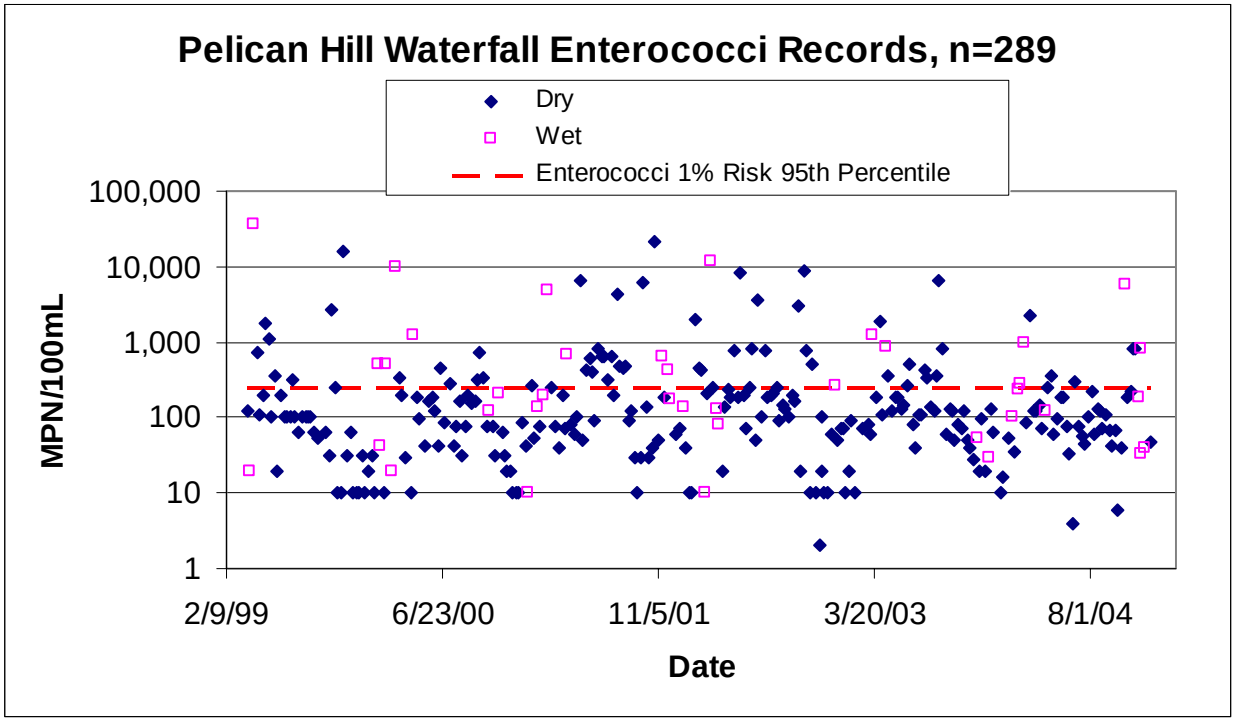


Figure A 14: Pelican Hill Waterfall enterococci data and corresponding cumulative frequency distribution



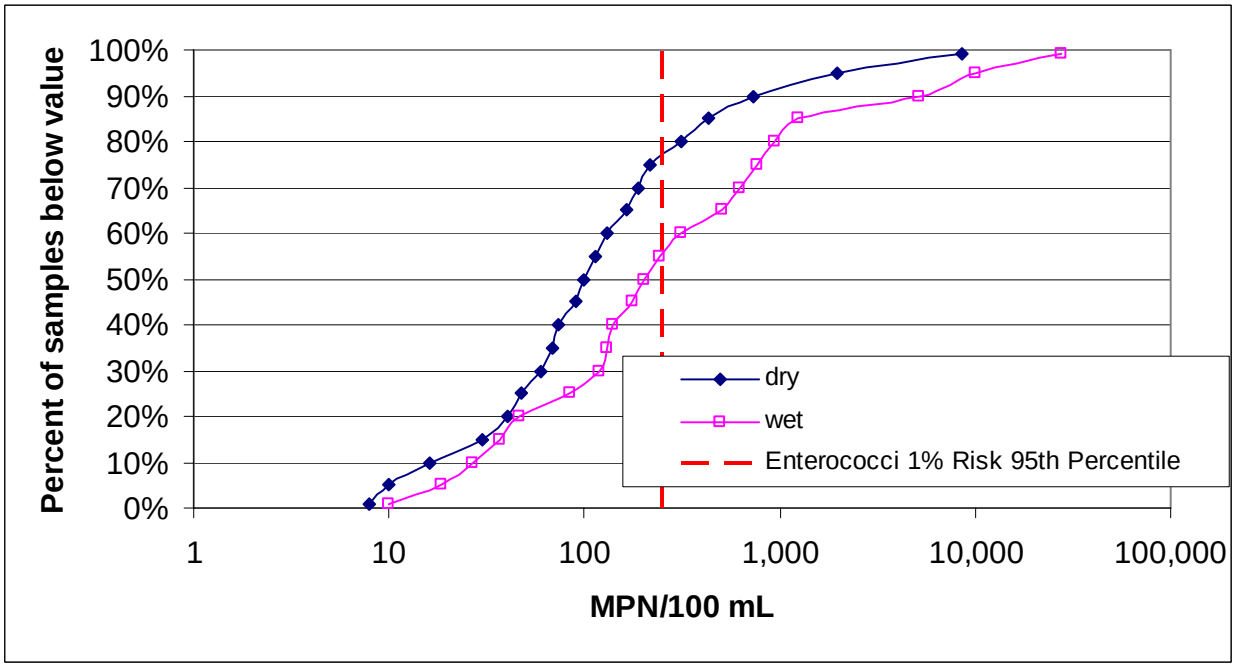
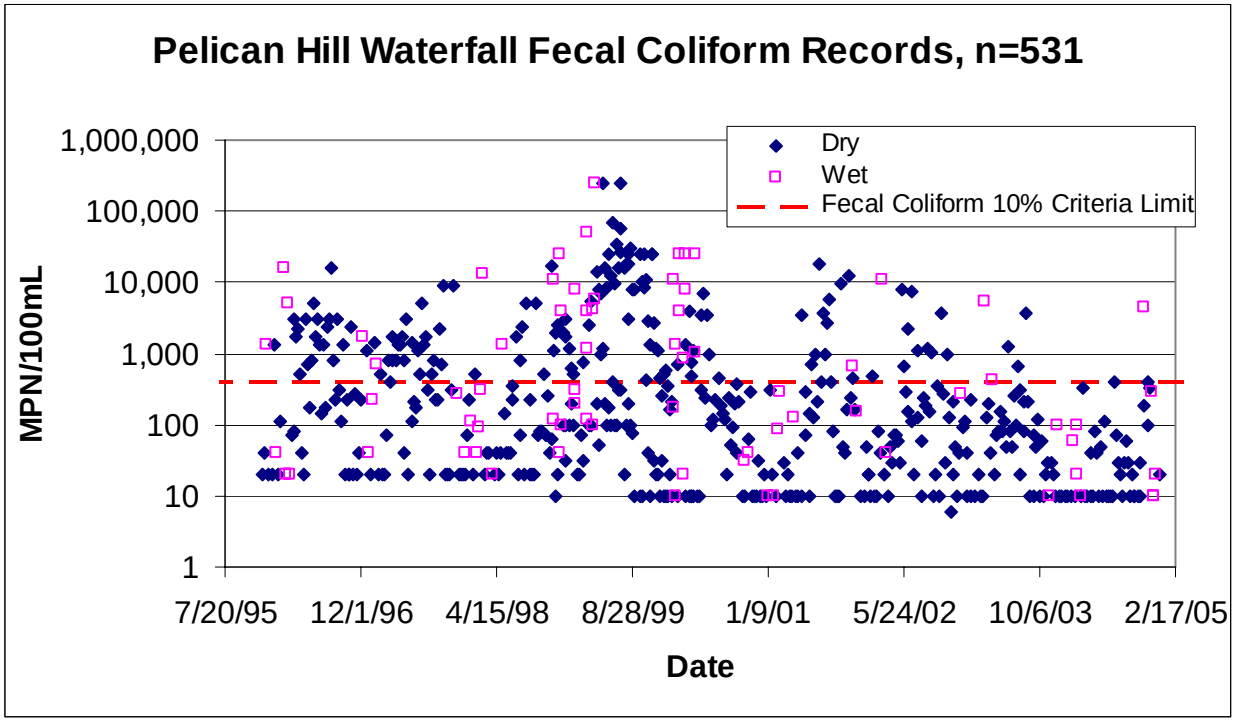


Figure A 15: Pelican Hill Waterfall fecal coliform data and corresponding cumulative frequency distribution



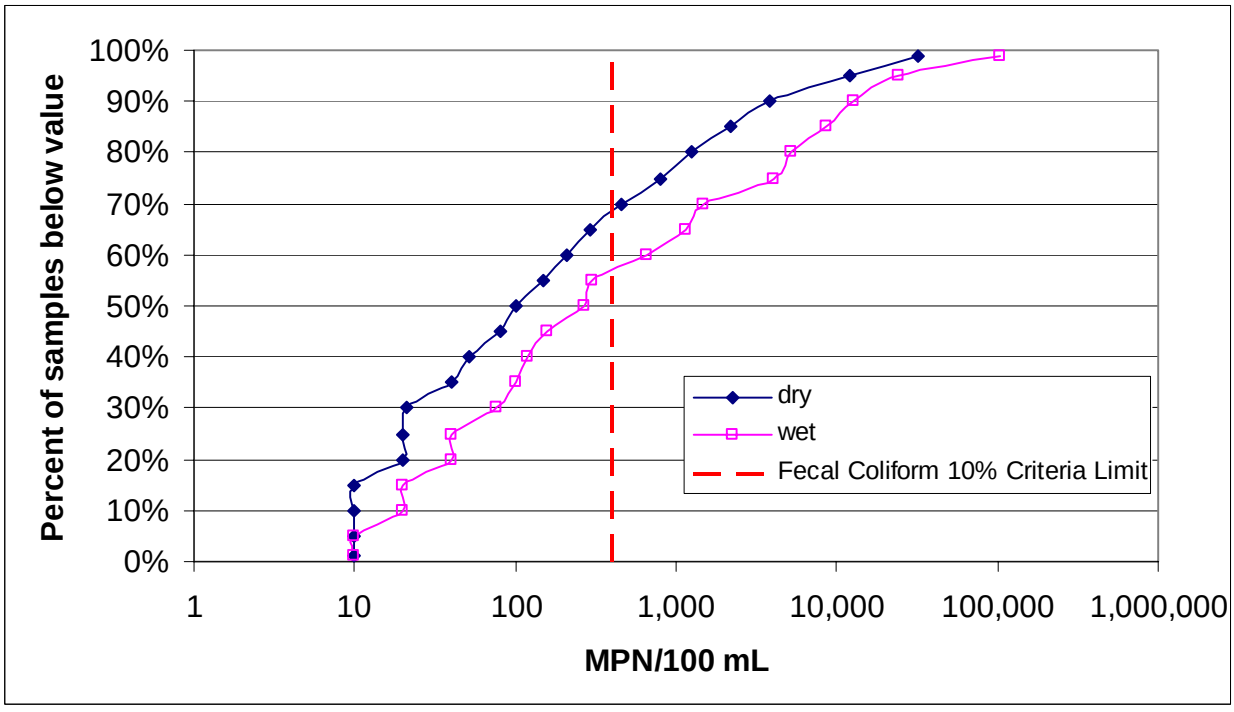
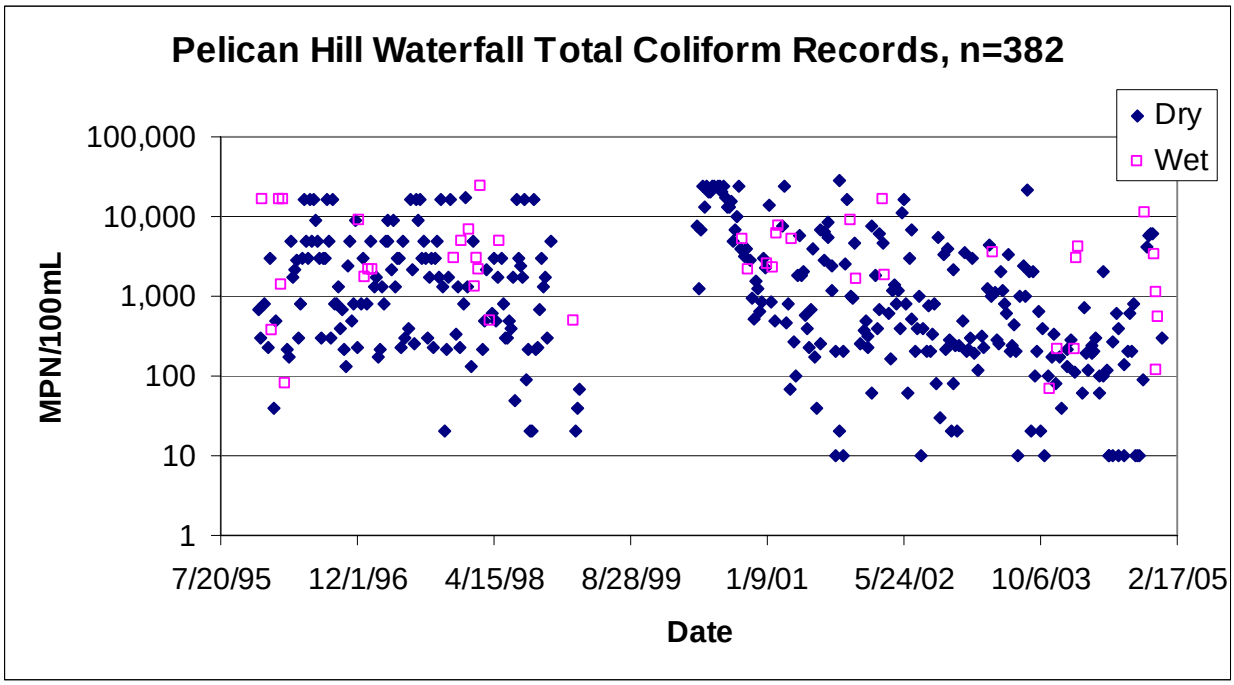


Figure A 16: Pelican Hill Waterfall total coliform data and corresponding cumulative frequency distribution



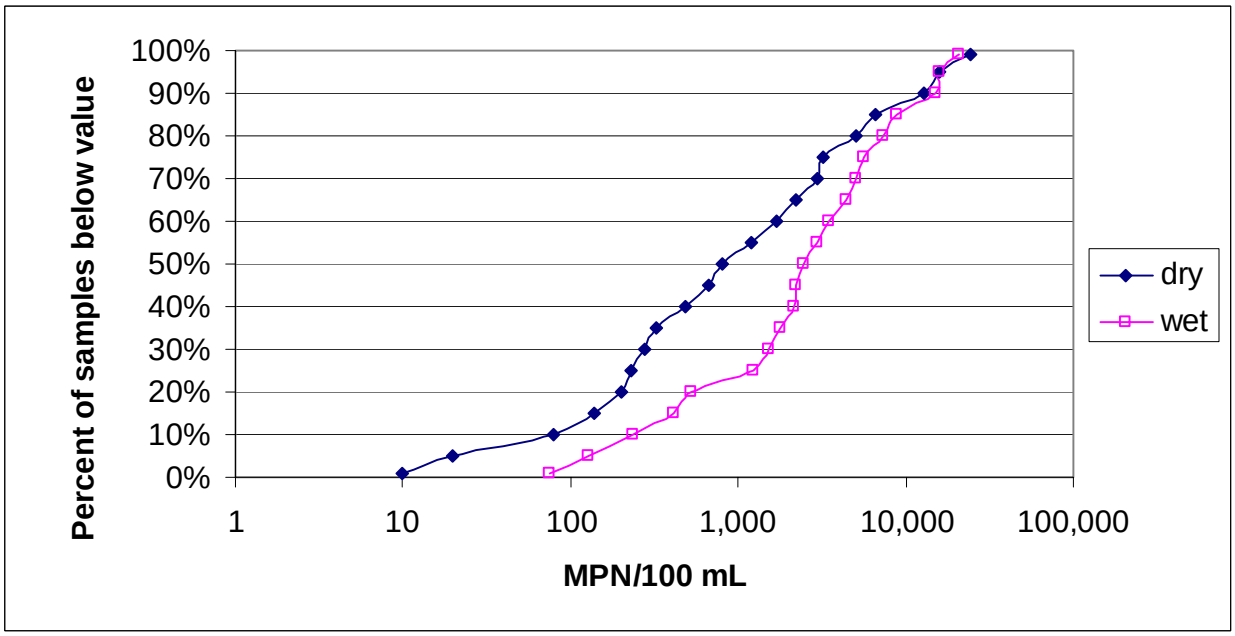


Figure A 17: Percentage of samples from Pelican Hill Waterfall which exceed thresholds, by month

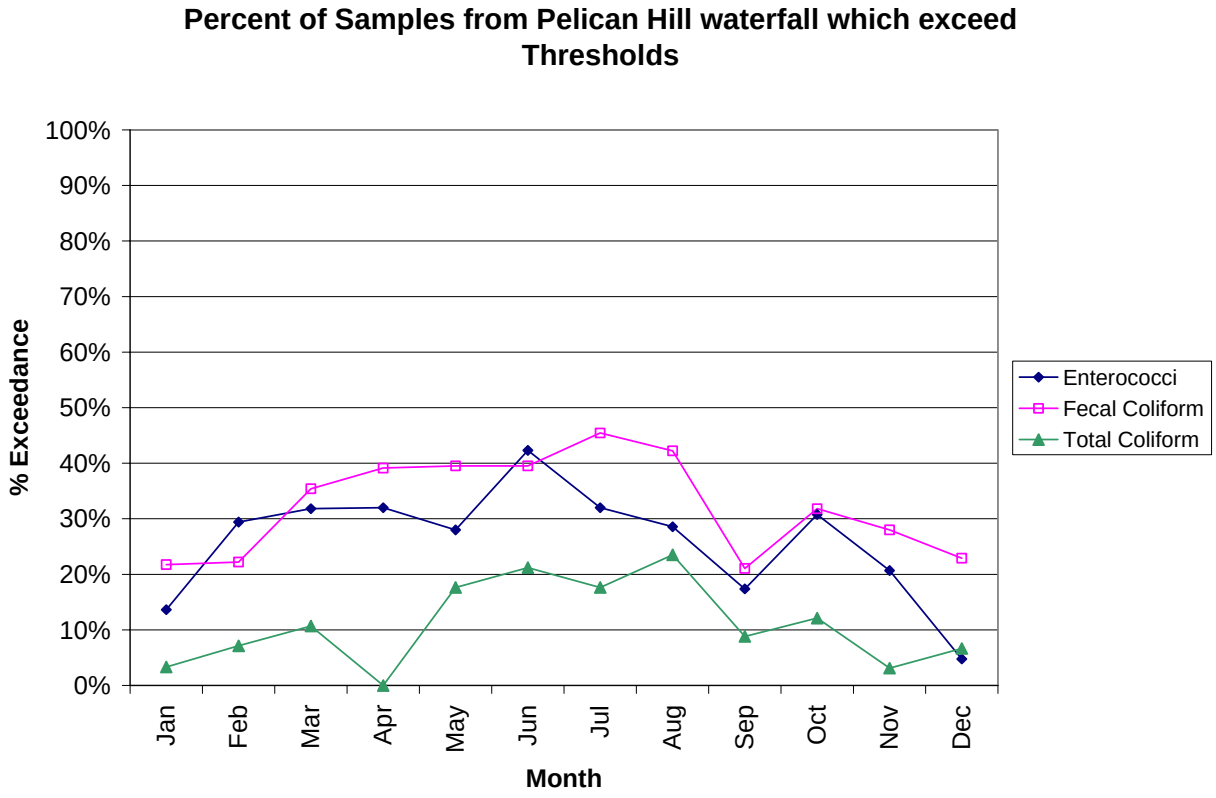
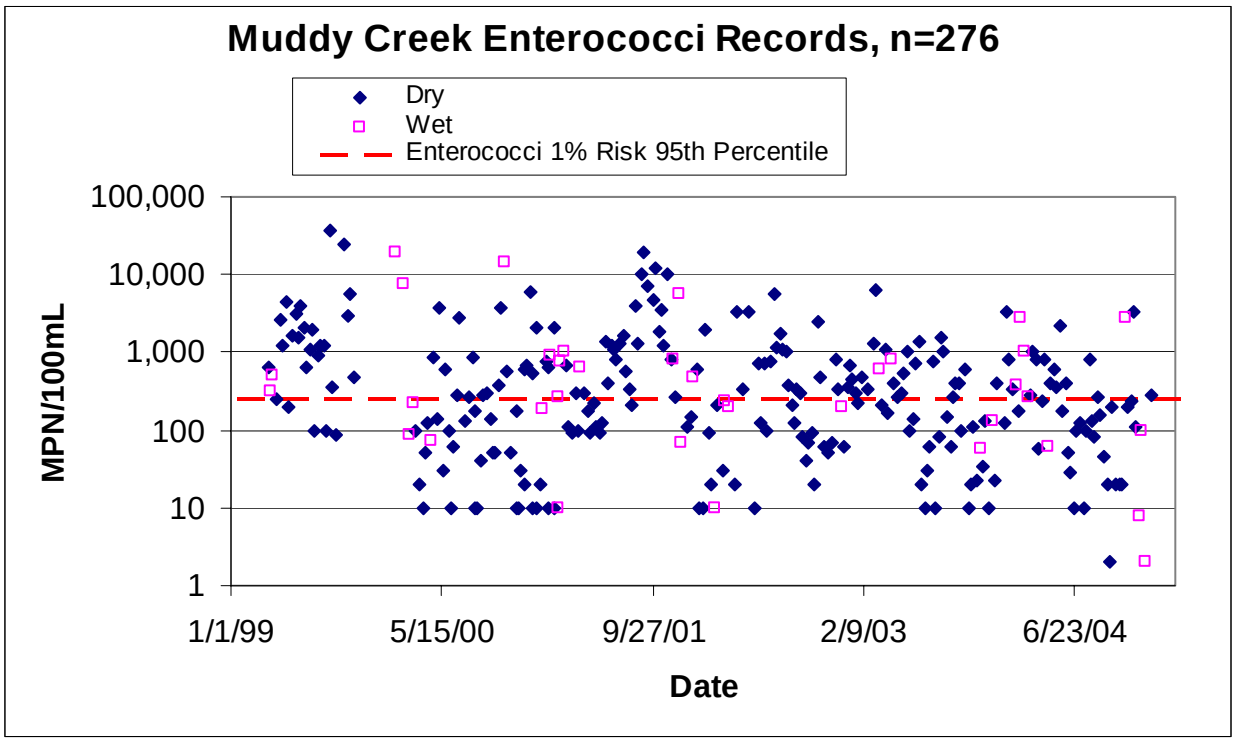


Figure A 18: Muddy Creek enterococci data and corresponding cumulative frequency distribution



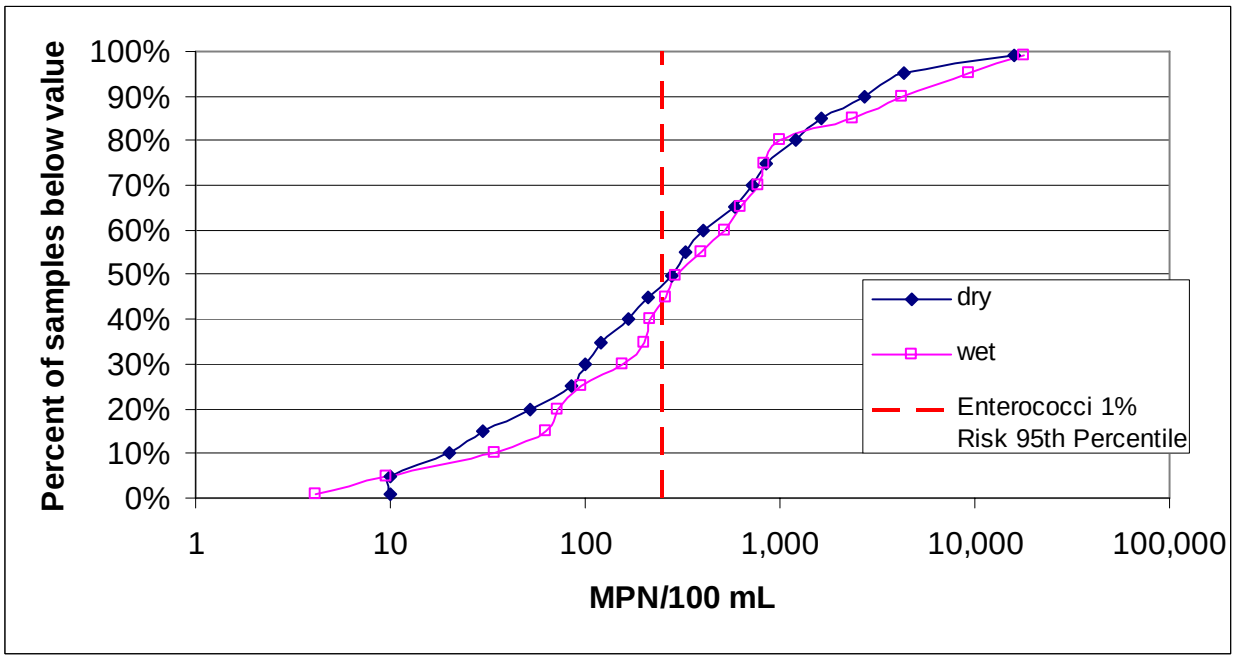
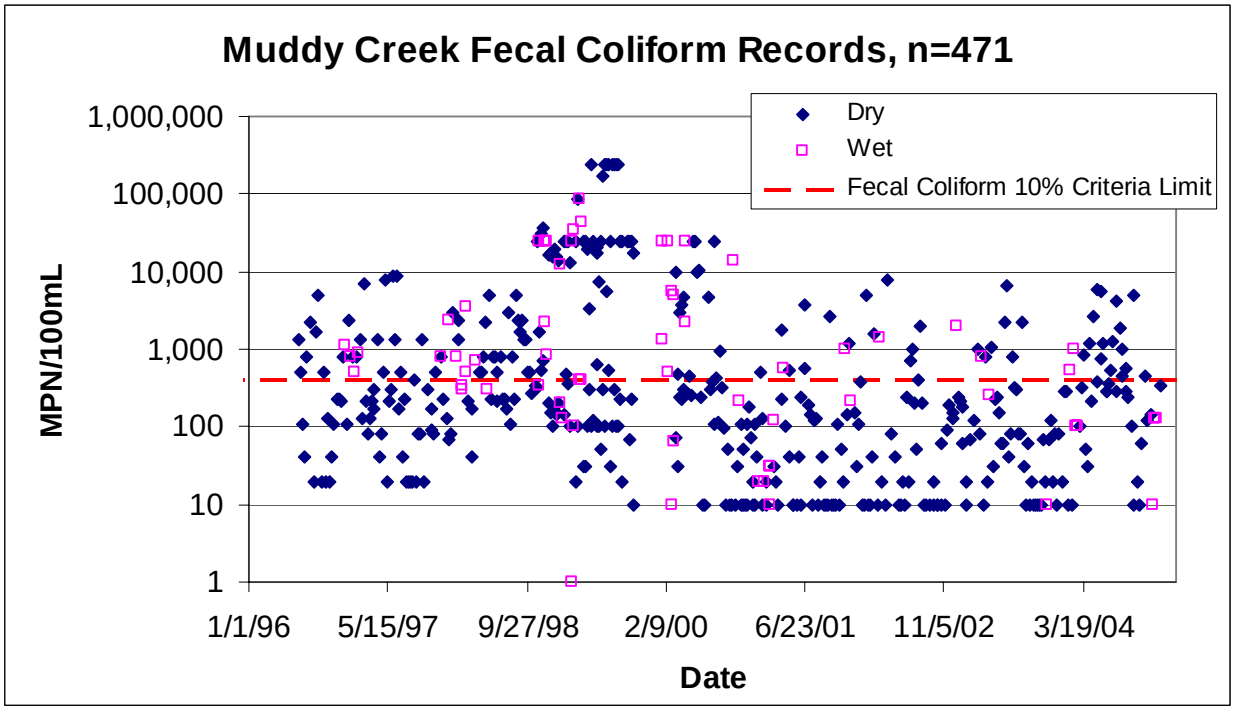


Figure A 19: Muddy Creek fecal coliform data and corresponding cumulative frequency distribution



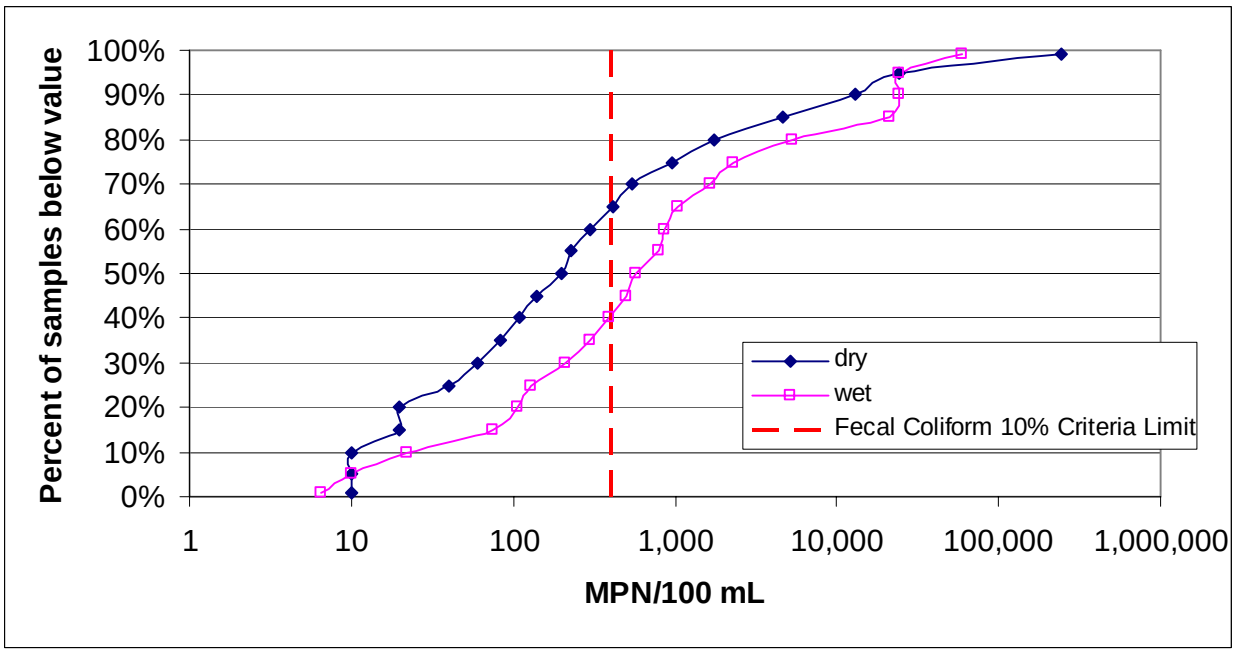
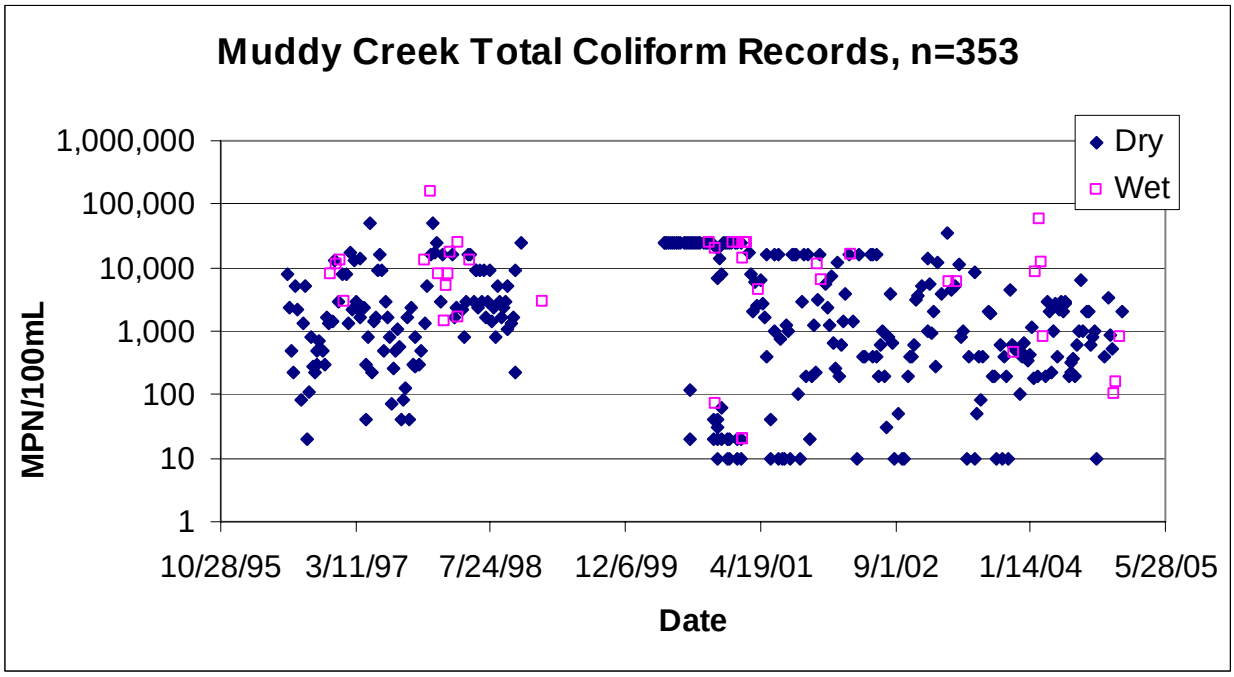


Figure A 20: Muddy Creek total coliform data and corresponding cumulative frequency distribution



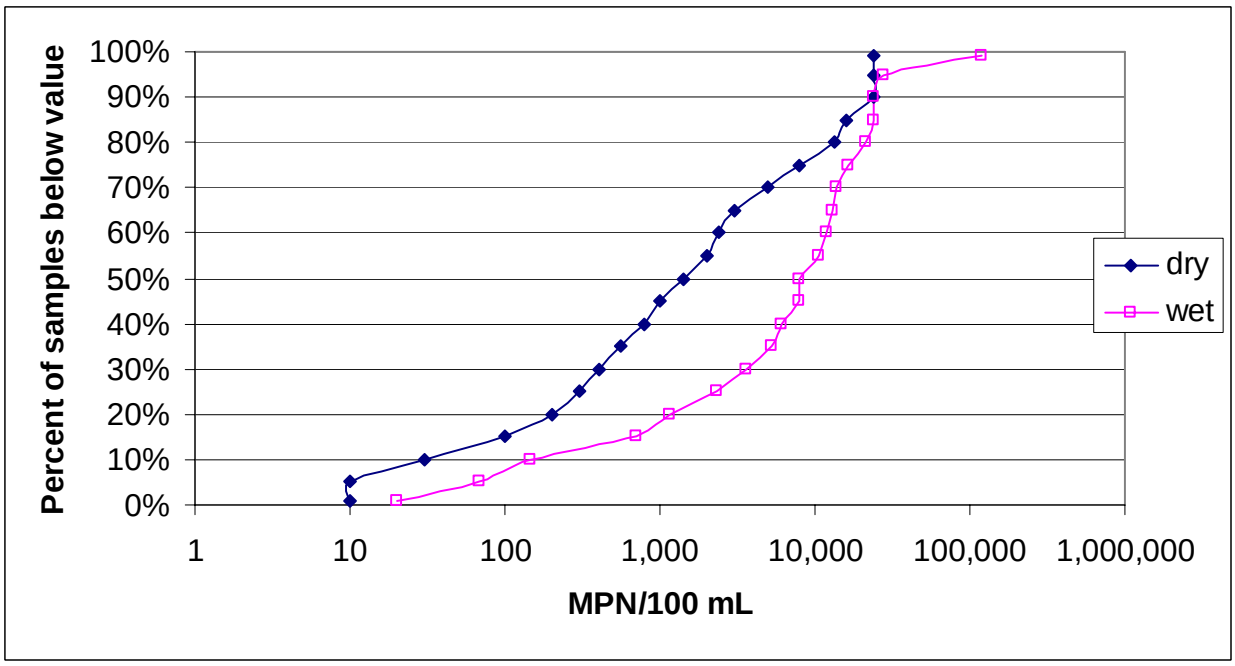


Figure A 21: Percentage of samples from Muddy Creek which exceed thresholds, by month

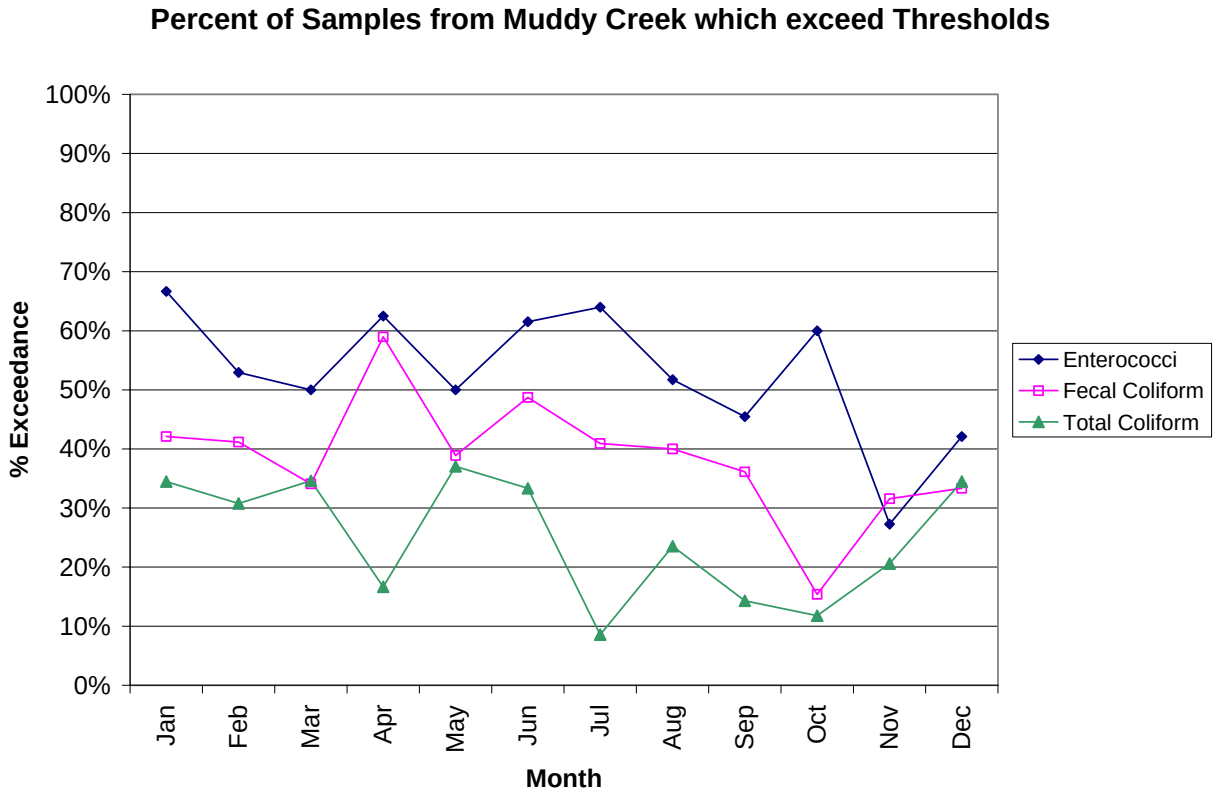
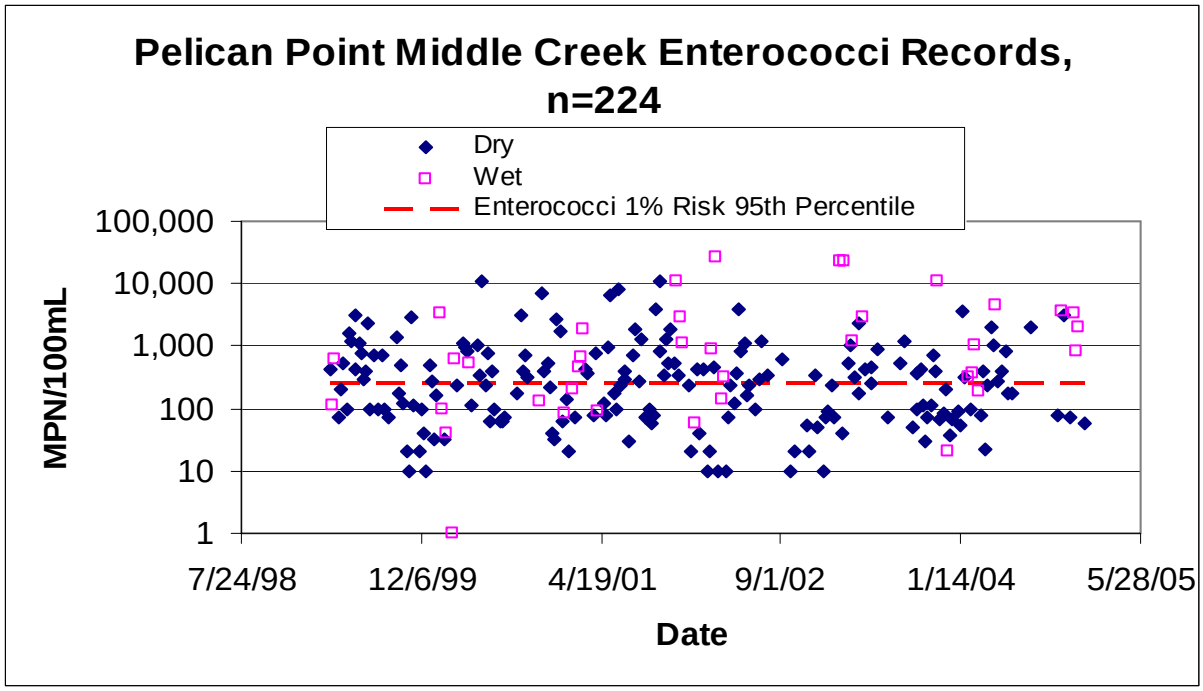


Figure A 22: Pelican Point Middle Creek enterococci data and corresponding cumulative frequency distribution



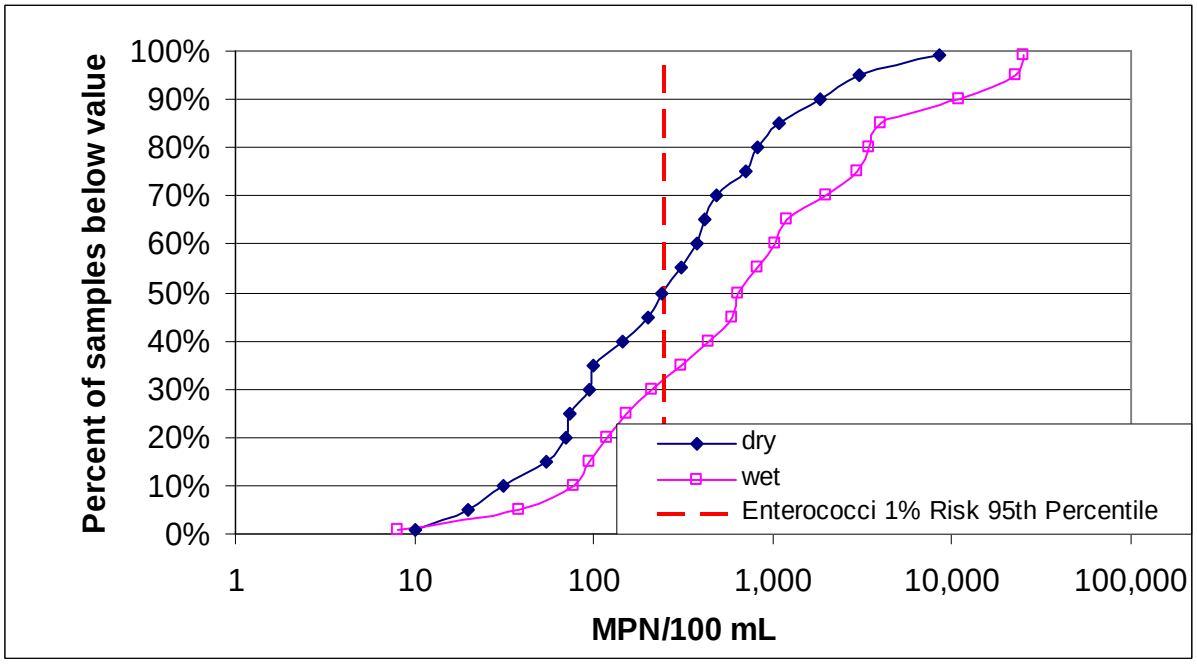
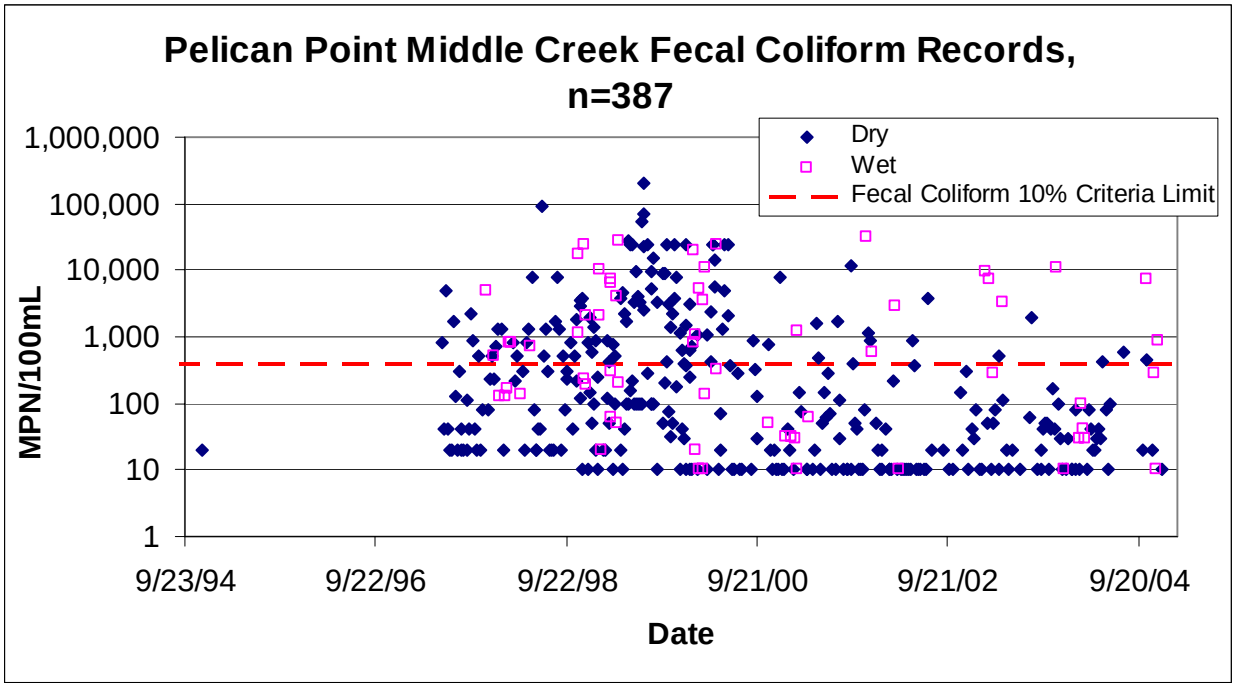


Figure A 23: Pelican Point Middle Creek fecal coliform data and corresponding cumulative frequency distribution



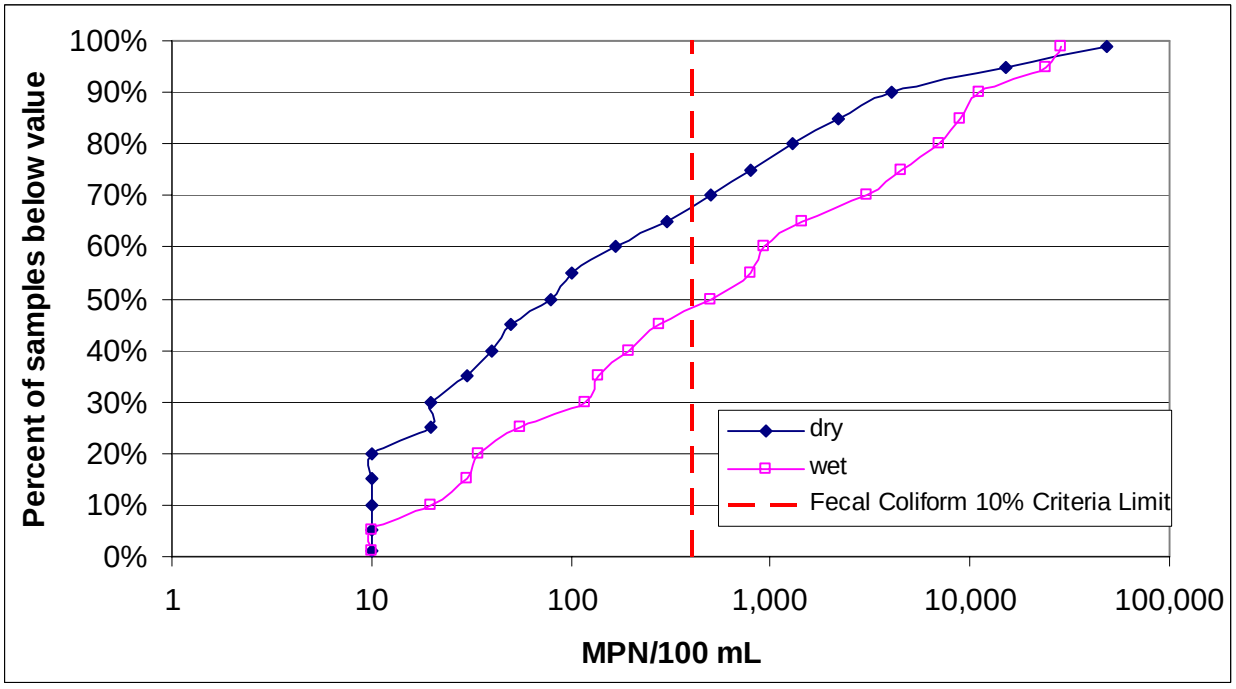
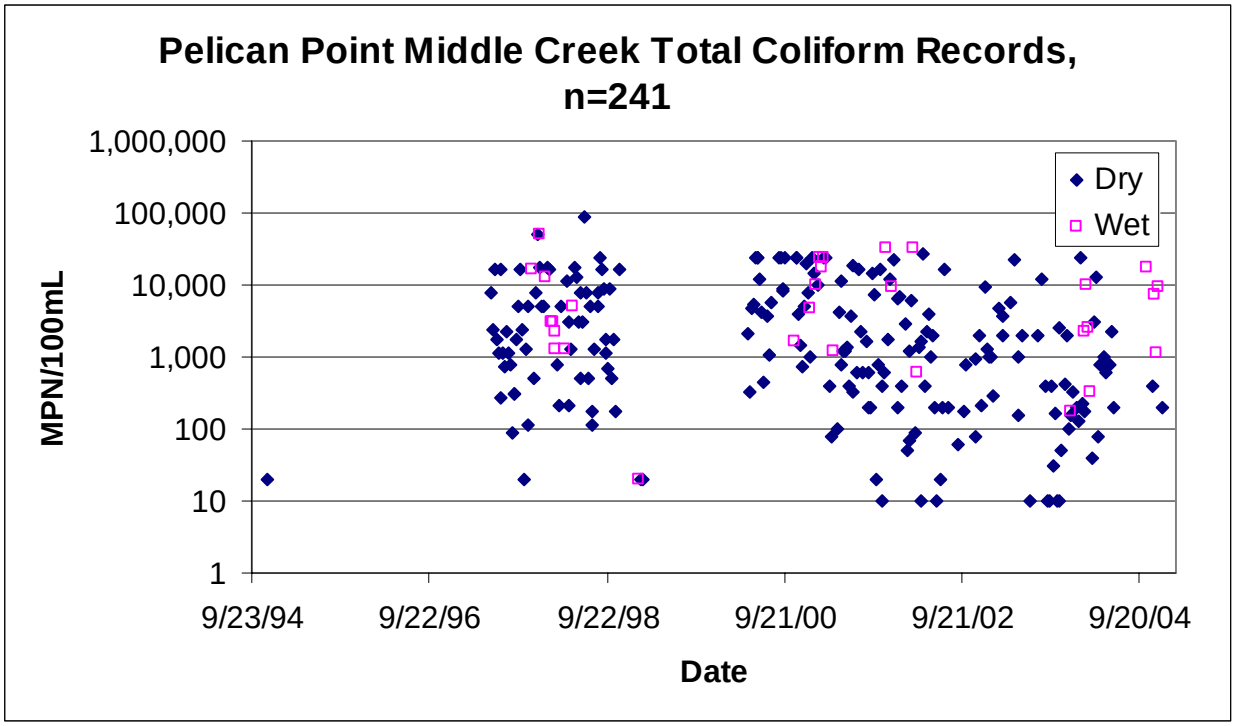


Figure A 24: Pelican Point Middle Creek total coliform data and corresponding cumulative frequency distribution



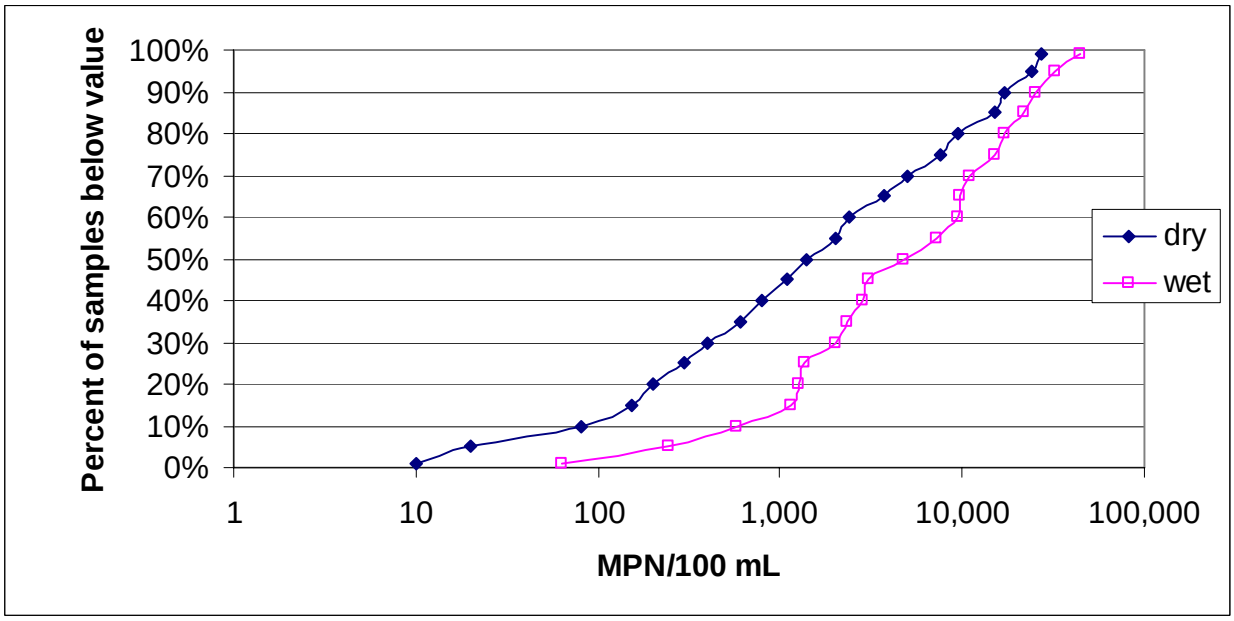


Figure A 25: Percentage of samples from Pelican Point Middle Creek which exceed thresholds, by month

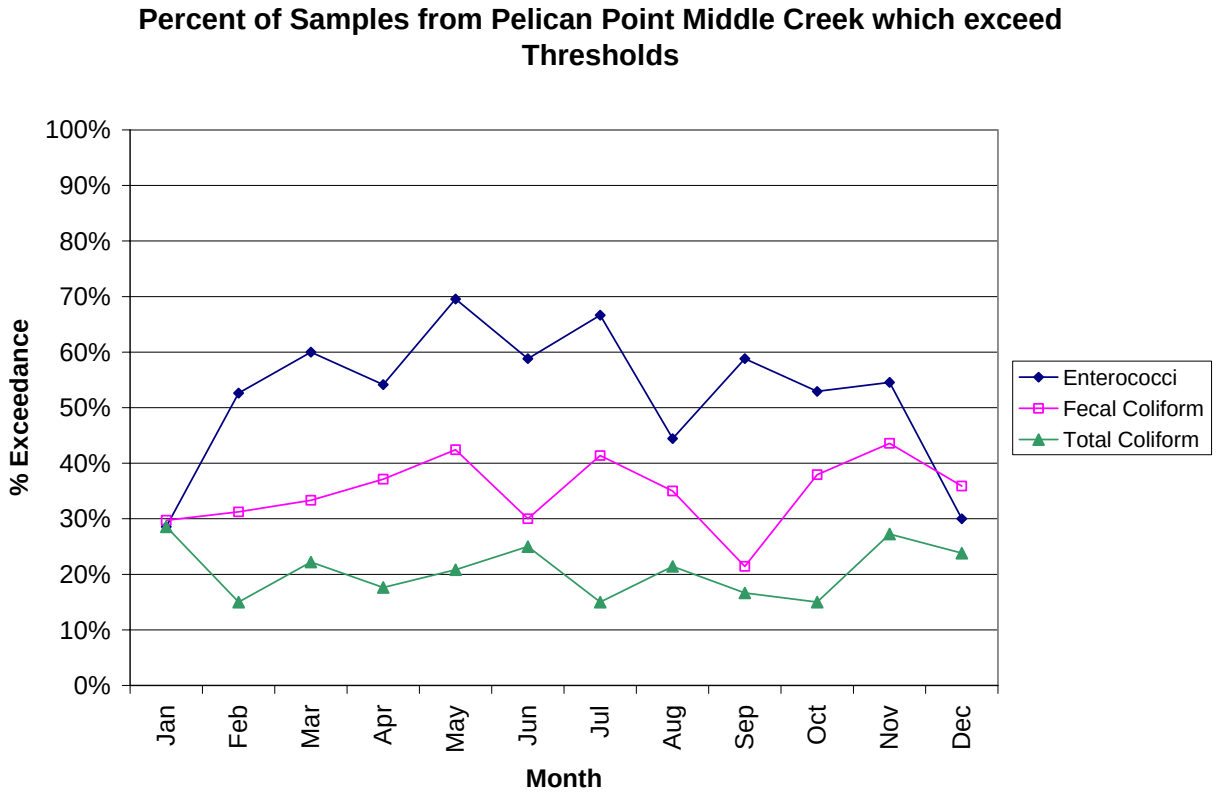
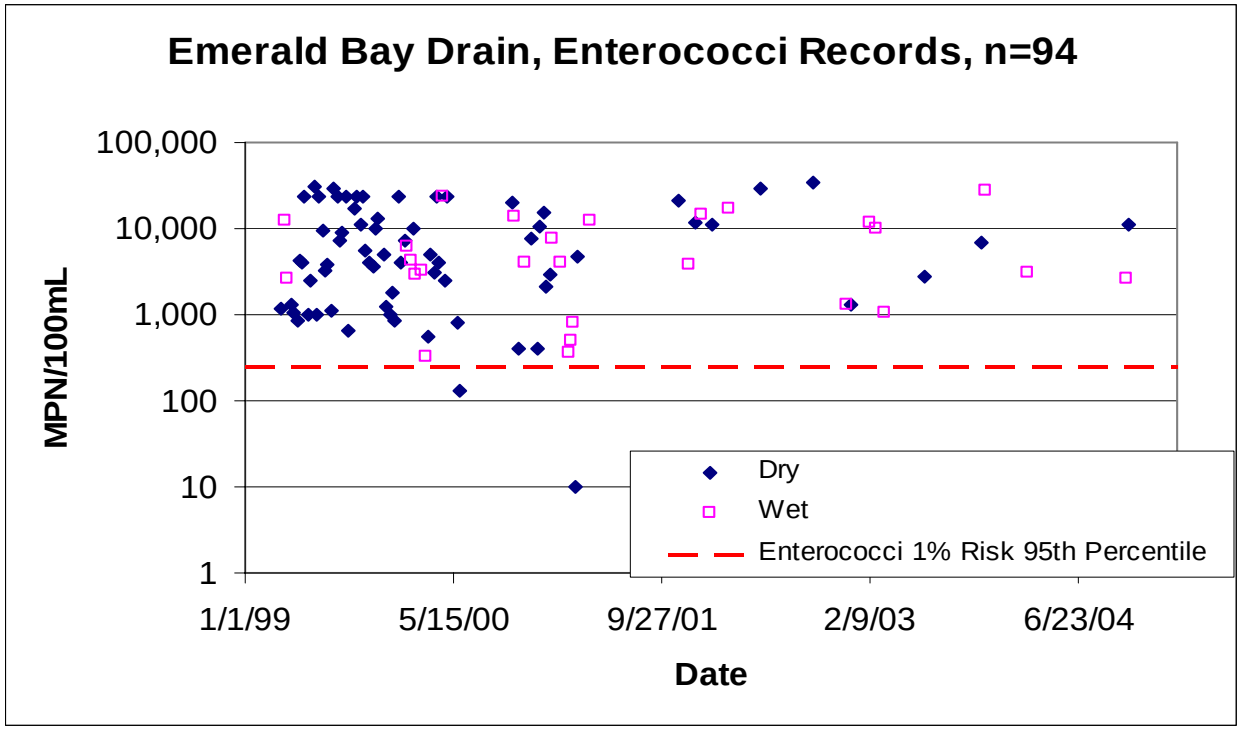


Figure A 26: Emerald Bay Drain enterococci data and corresponding cumulative frequency distribution



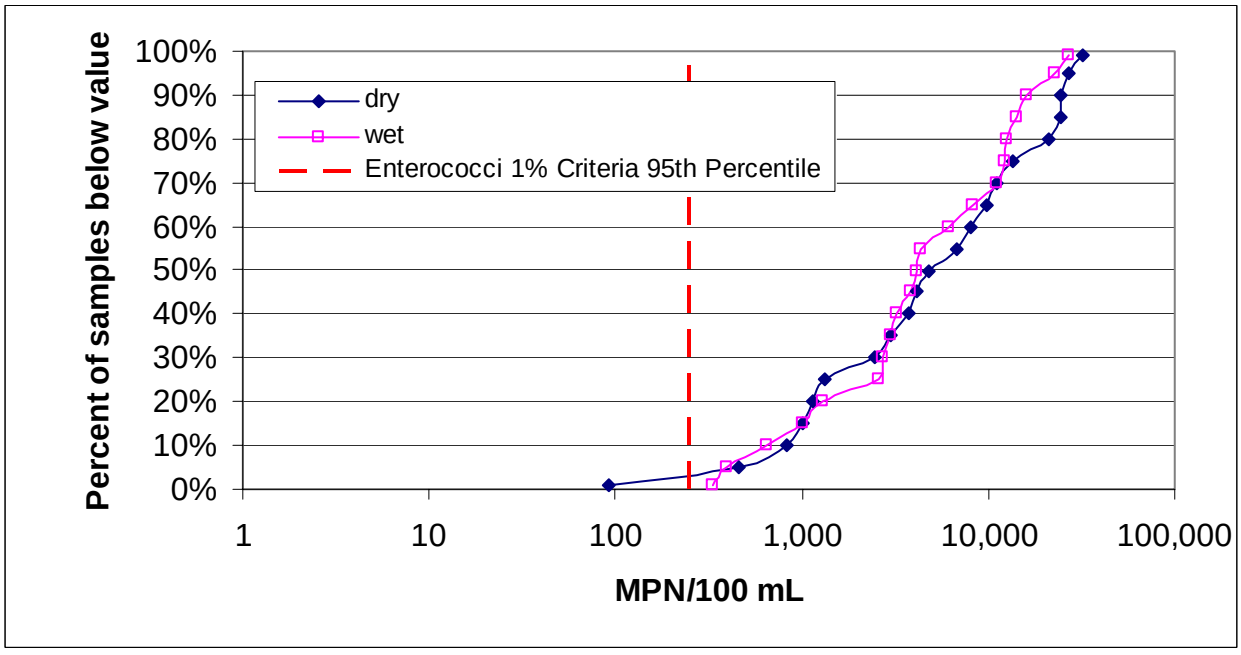
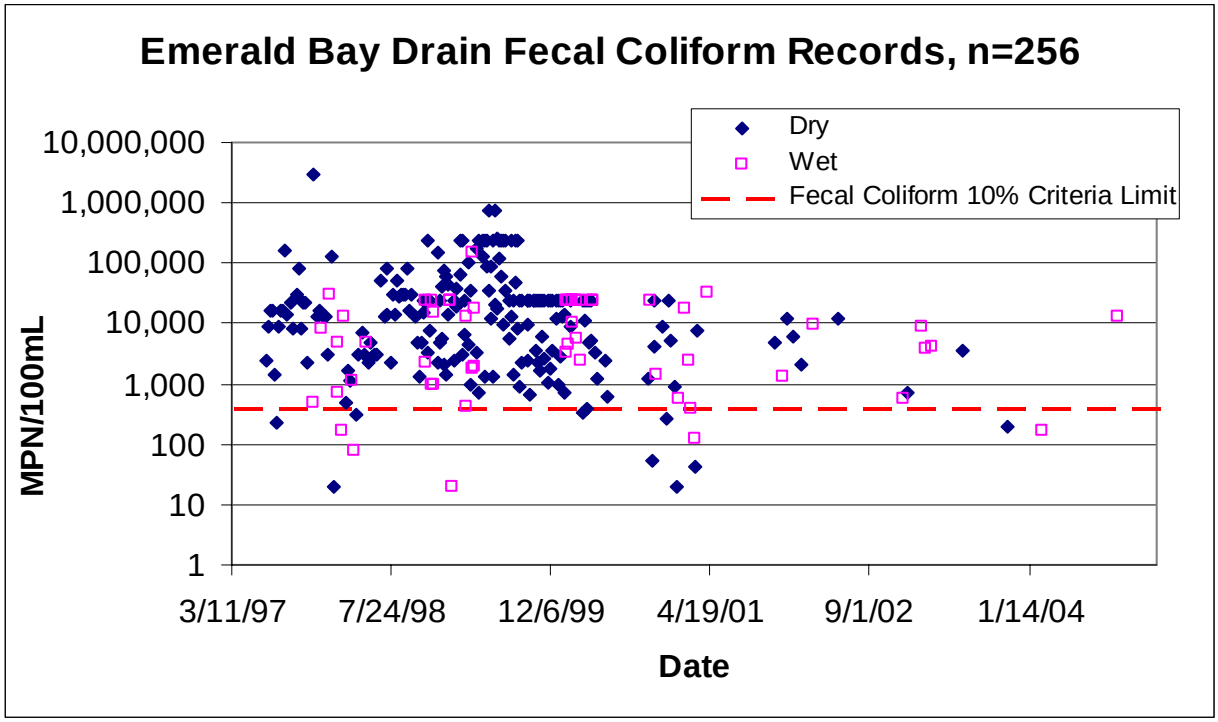


Figure A 27: Emerald Bay Drain fecal coliform data and corresponding cumulative frequency distribution



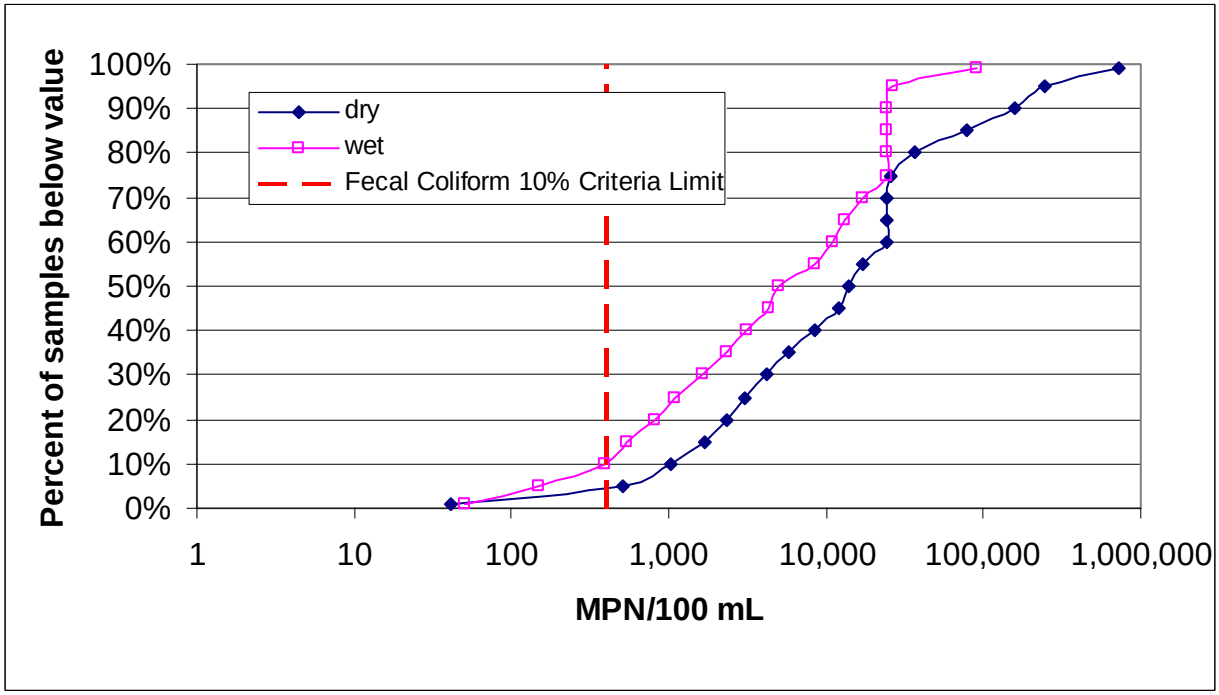
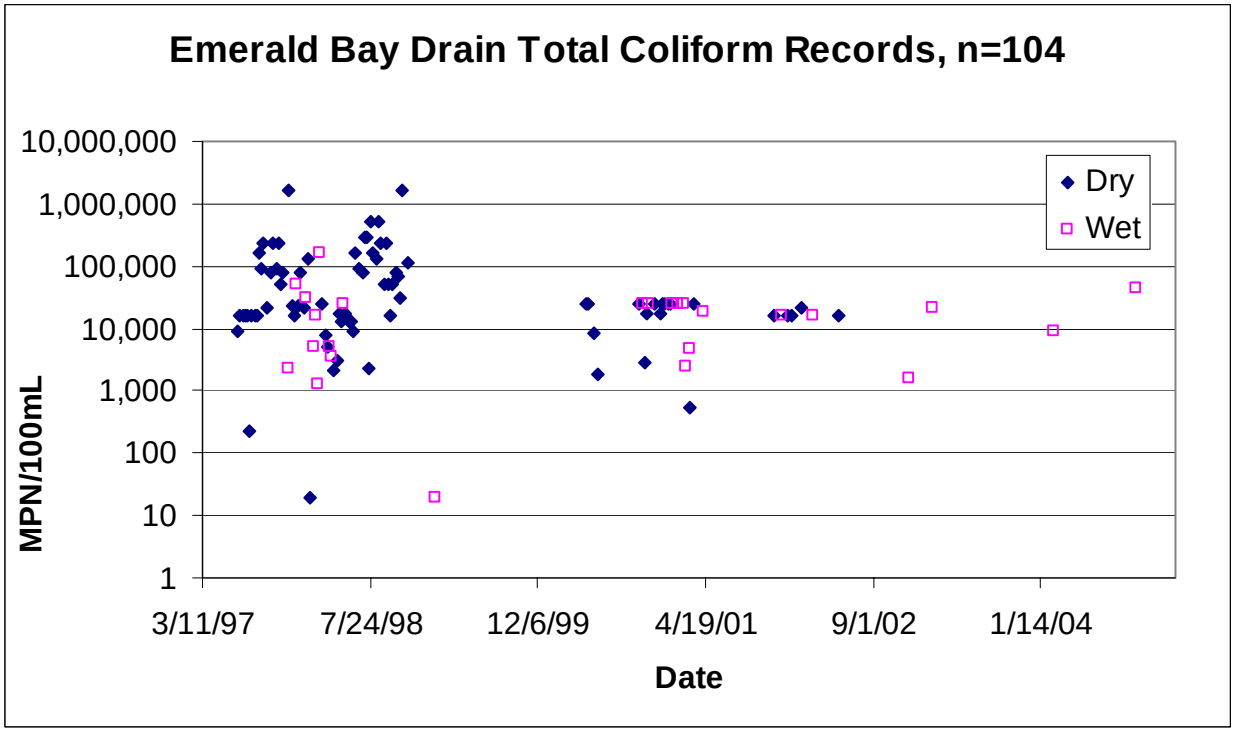


Figure A 28: Emerald Bay Drain total coliform data and corresponding cumulative frequency distribution



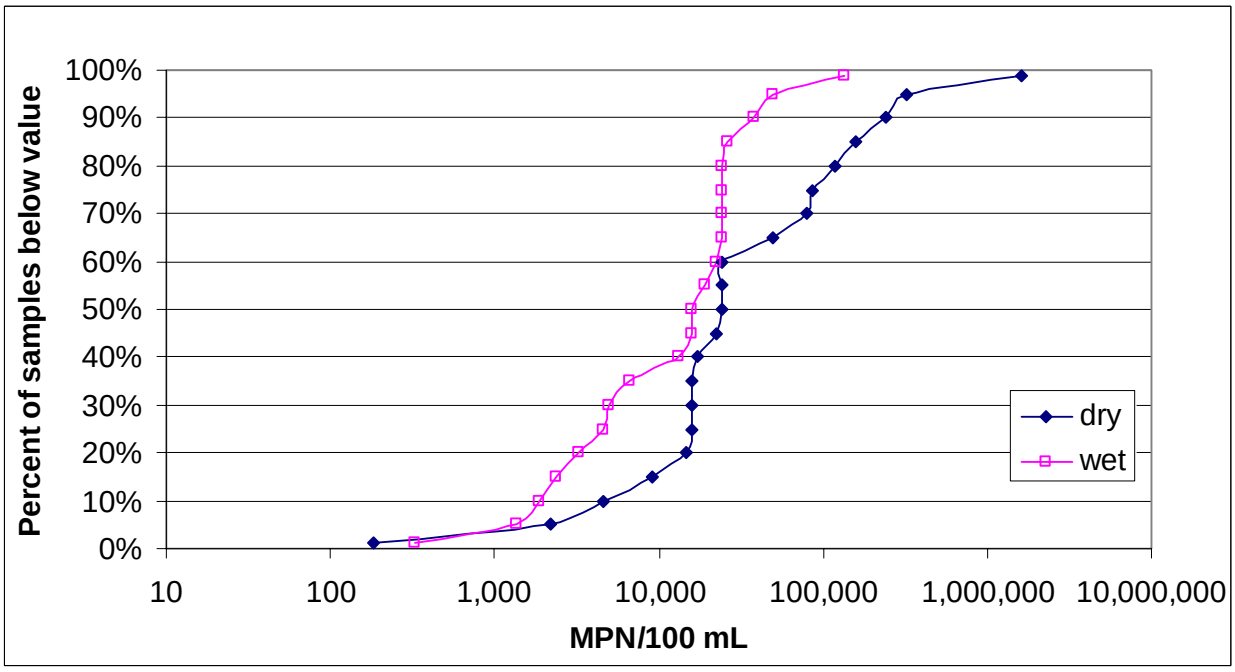


Figure A 29: Percentage of samples from the Emerald Bay Drain which exceed thresholds, by month

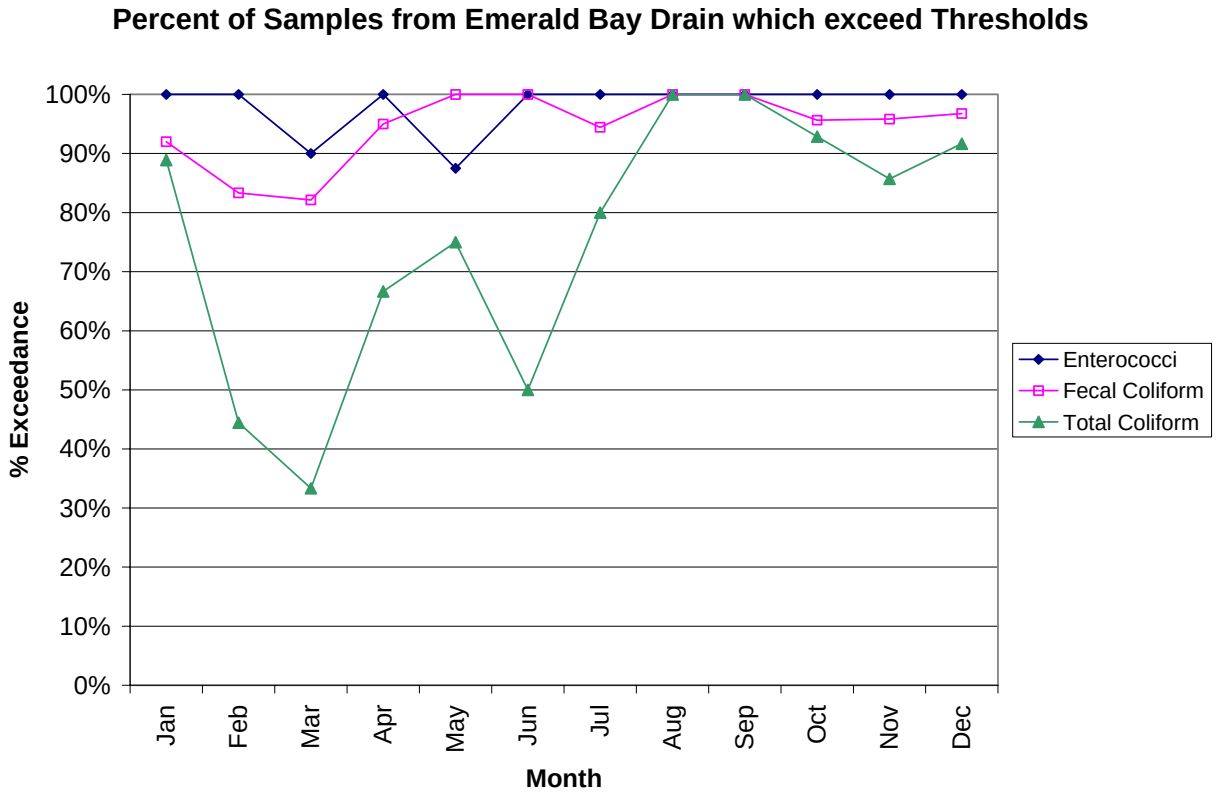
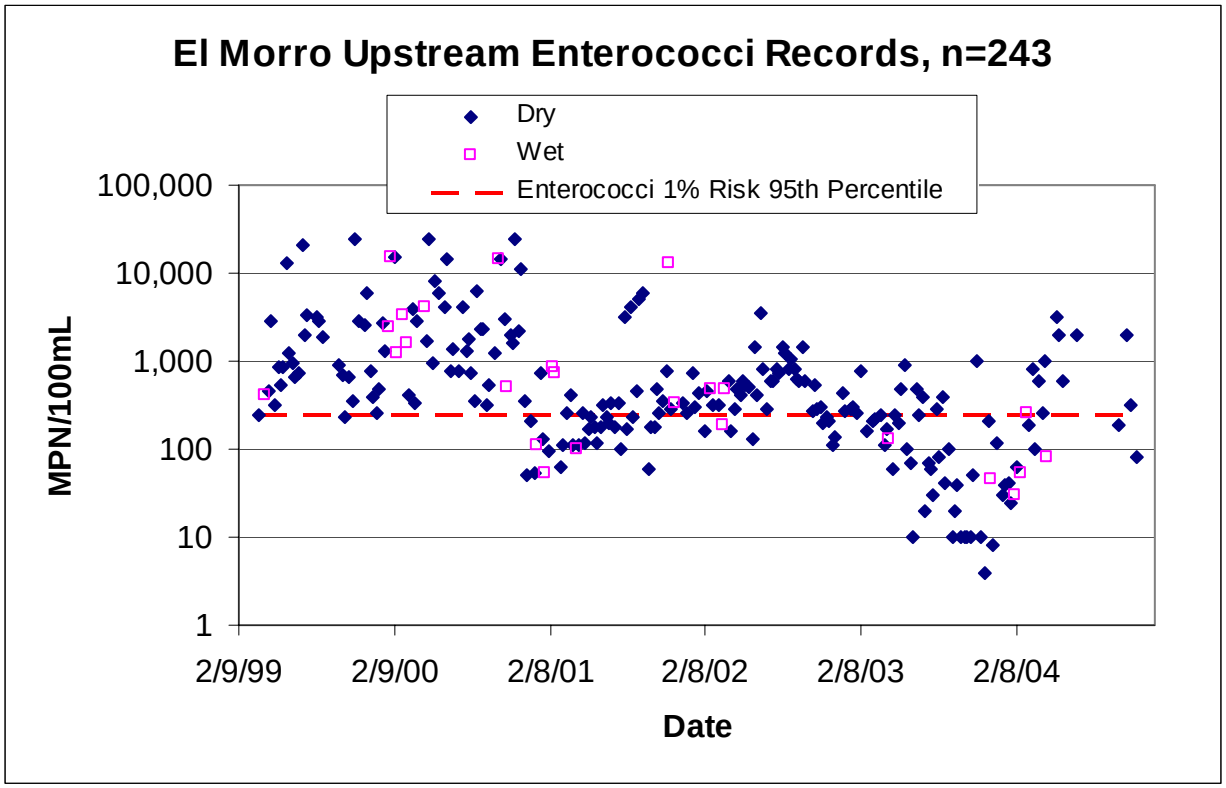


Figure A 30: El Morro Creek Upstream enterococci data and corresponding cumulative frequency distribution



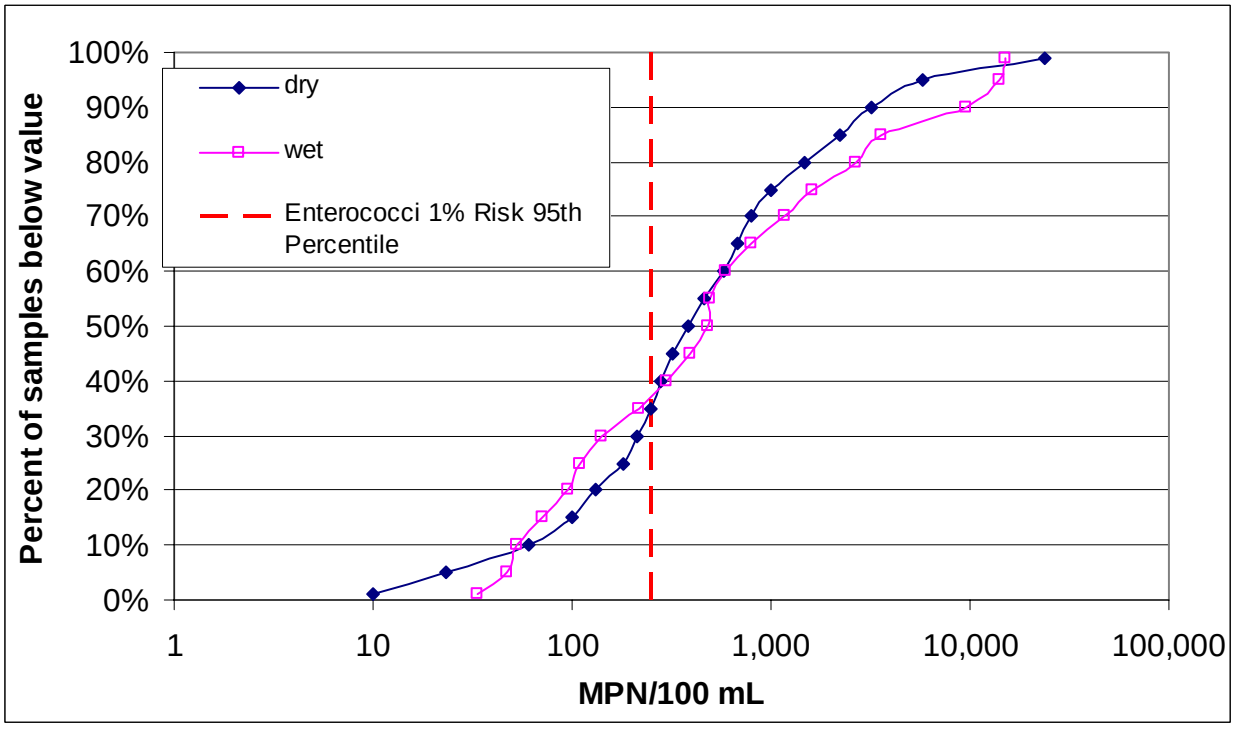
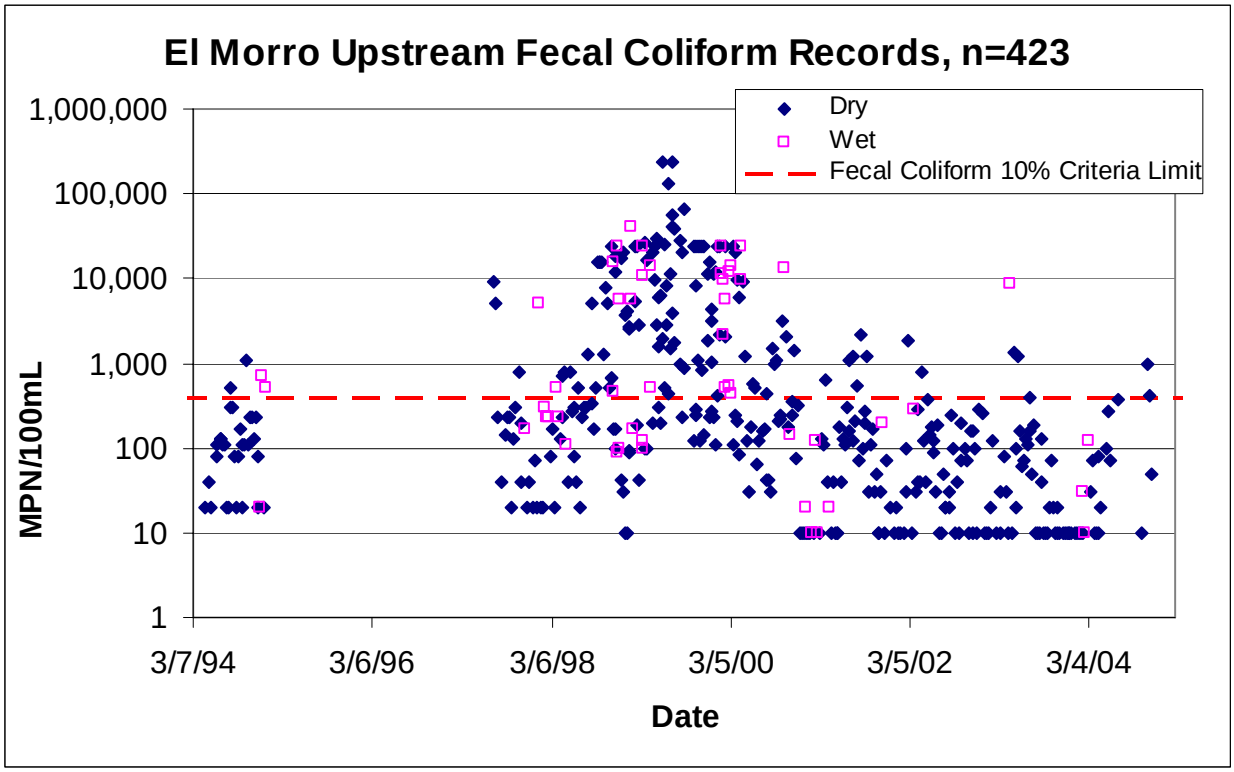


Figure A 31: El Morro Creek Upstream fecal coliform data and corresponding cumulative frequency distribution



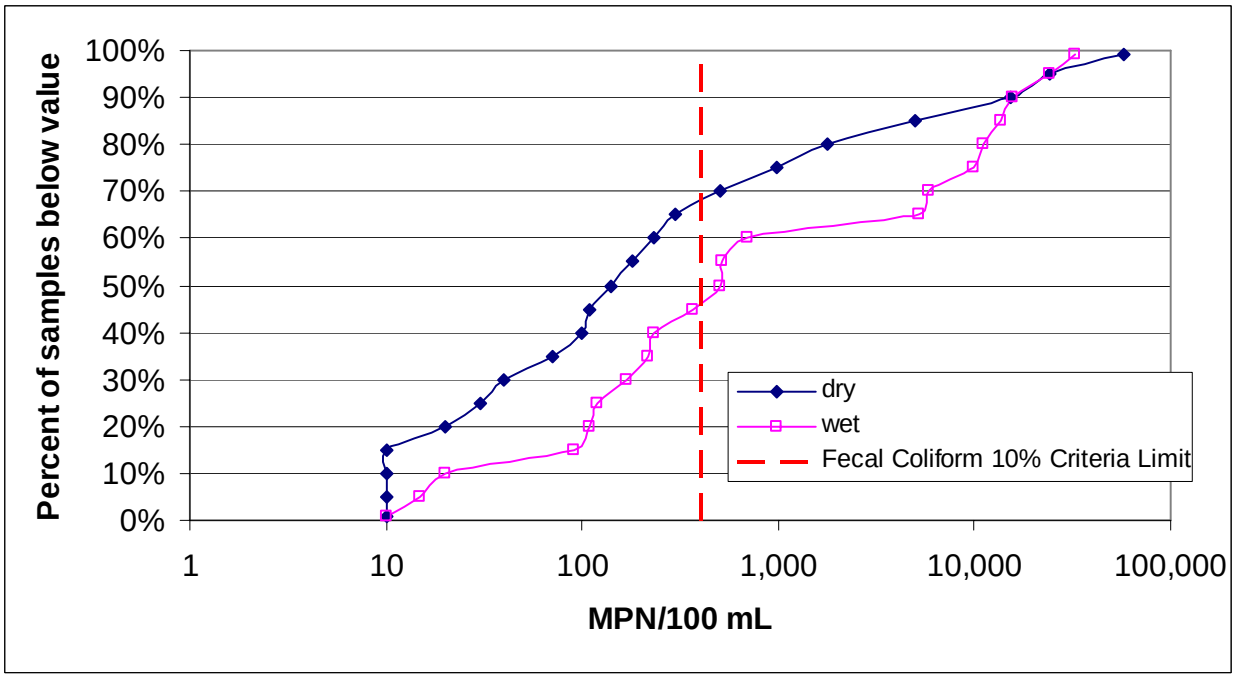
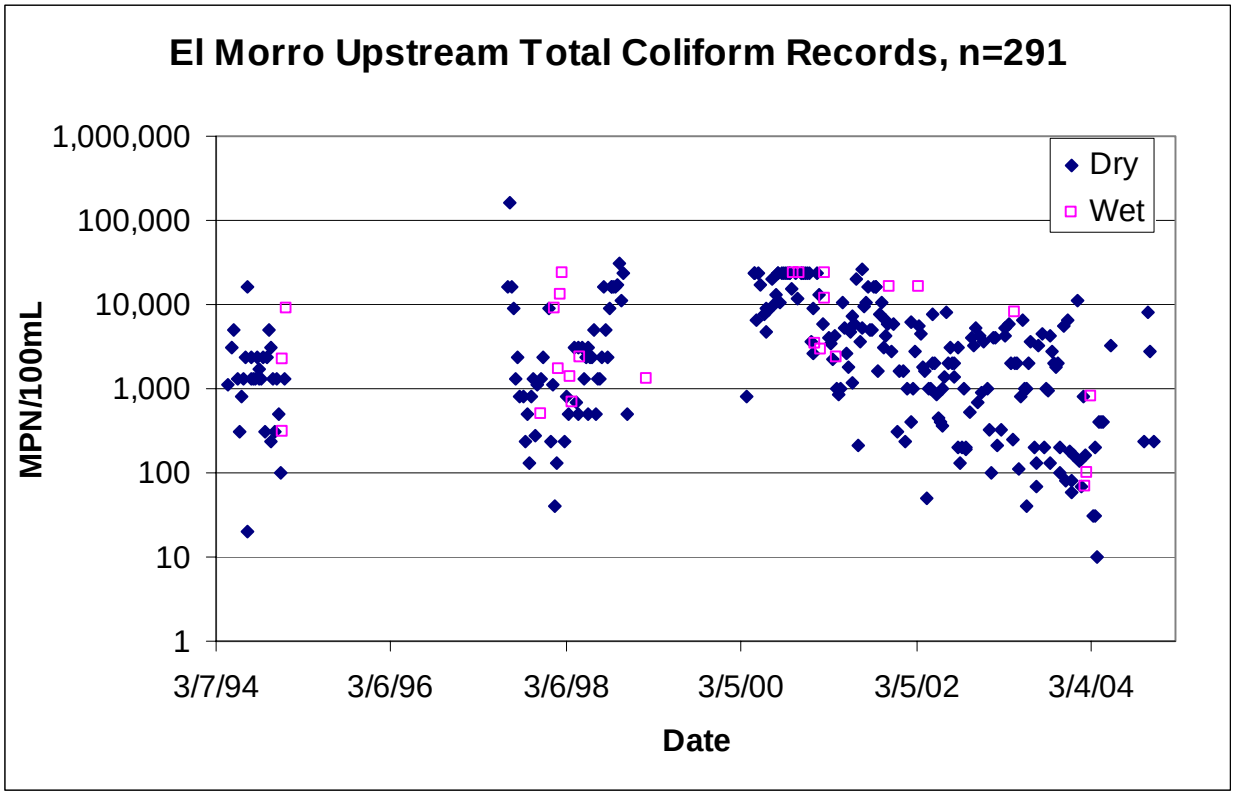


Figure A 32: El Morro Creek Upstream total coliform data and corresponding cumulative frequency distribution



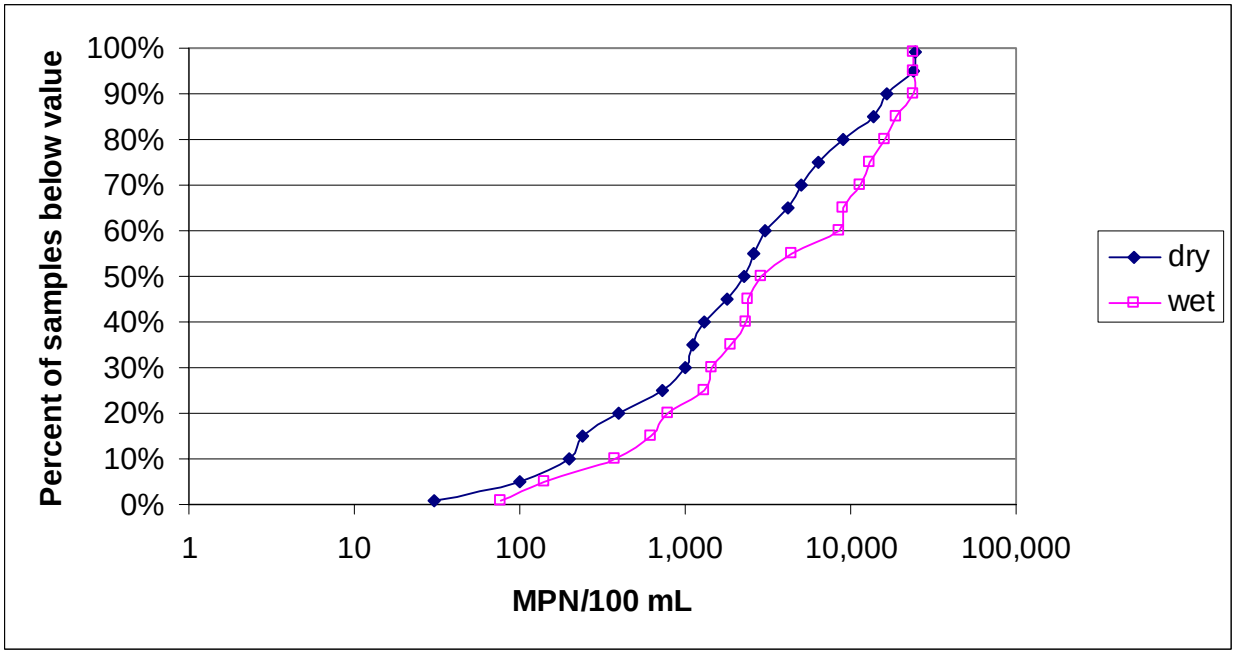


Figure A 33: Percentage of samples from El Morro Creek Upstream which exceed thresholds, by month

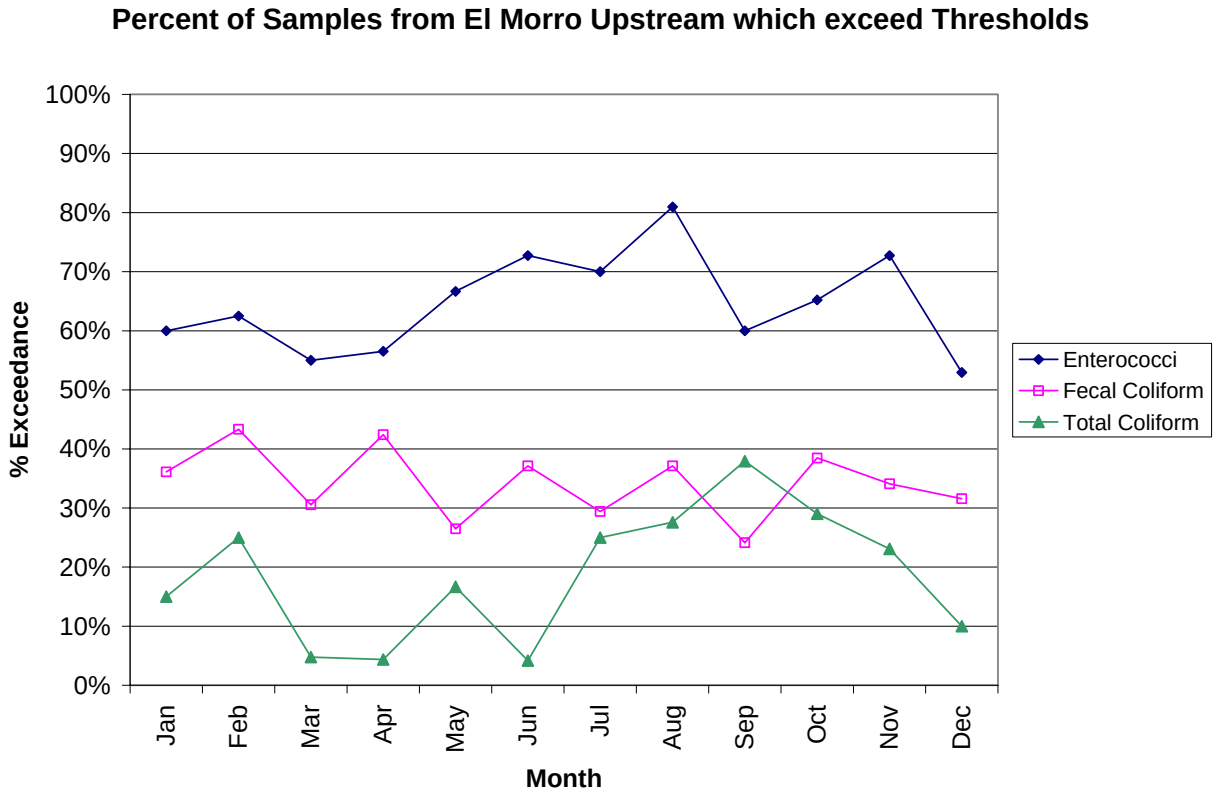
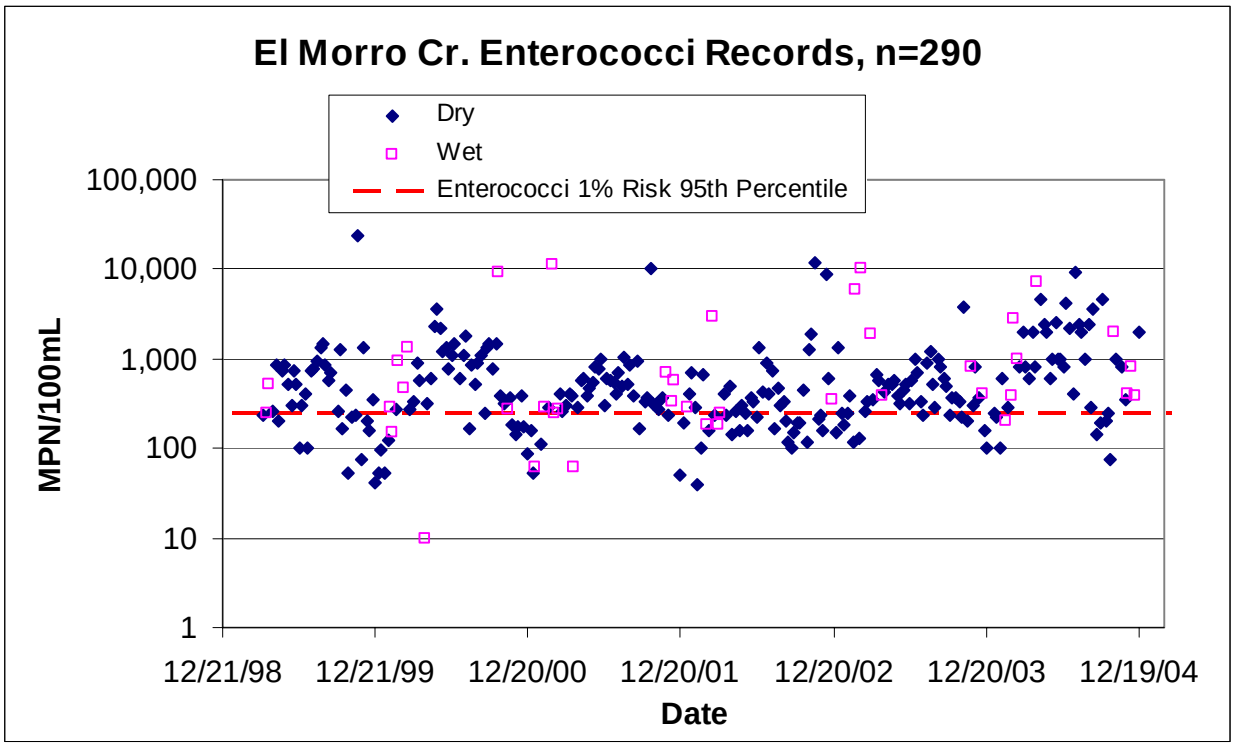


Figure A 34: El Morro Creek enterococci data and corresponding cumulative frequency distribution



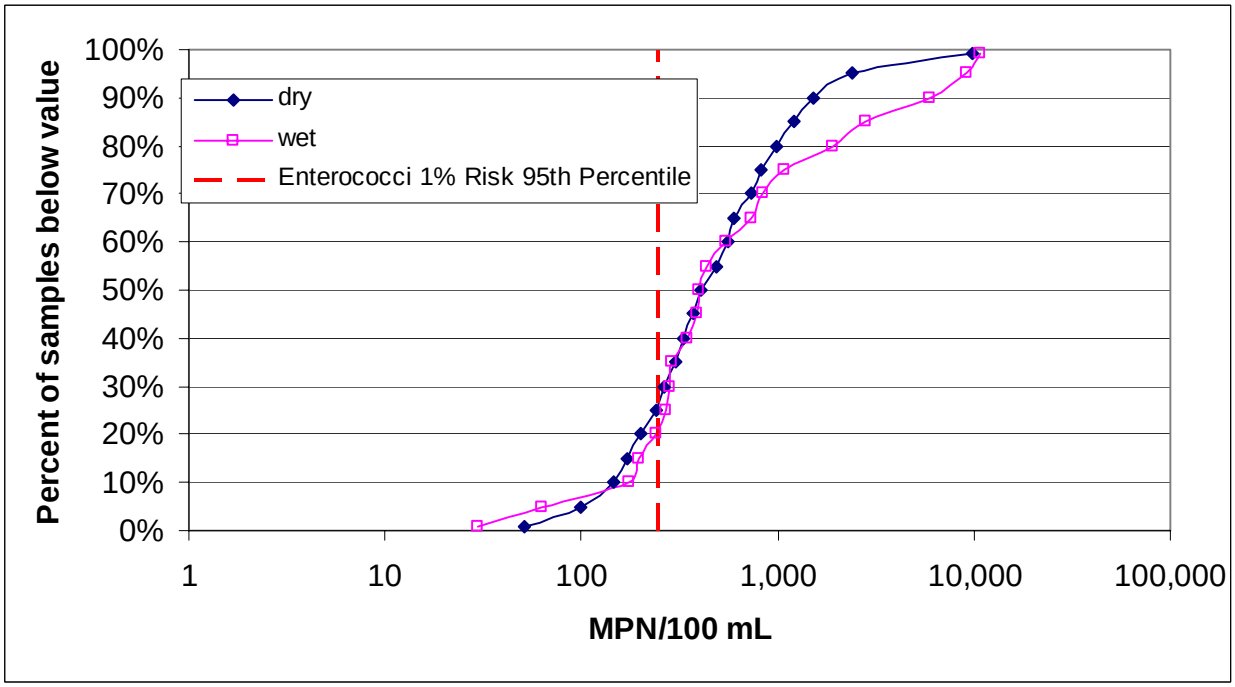
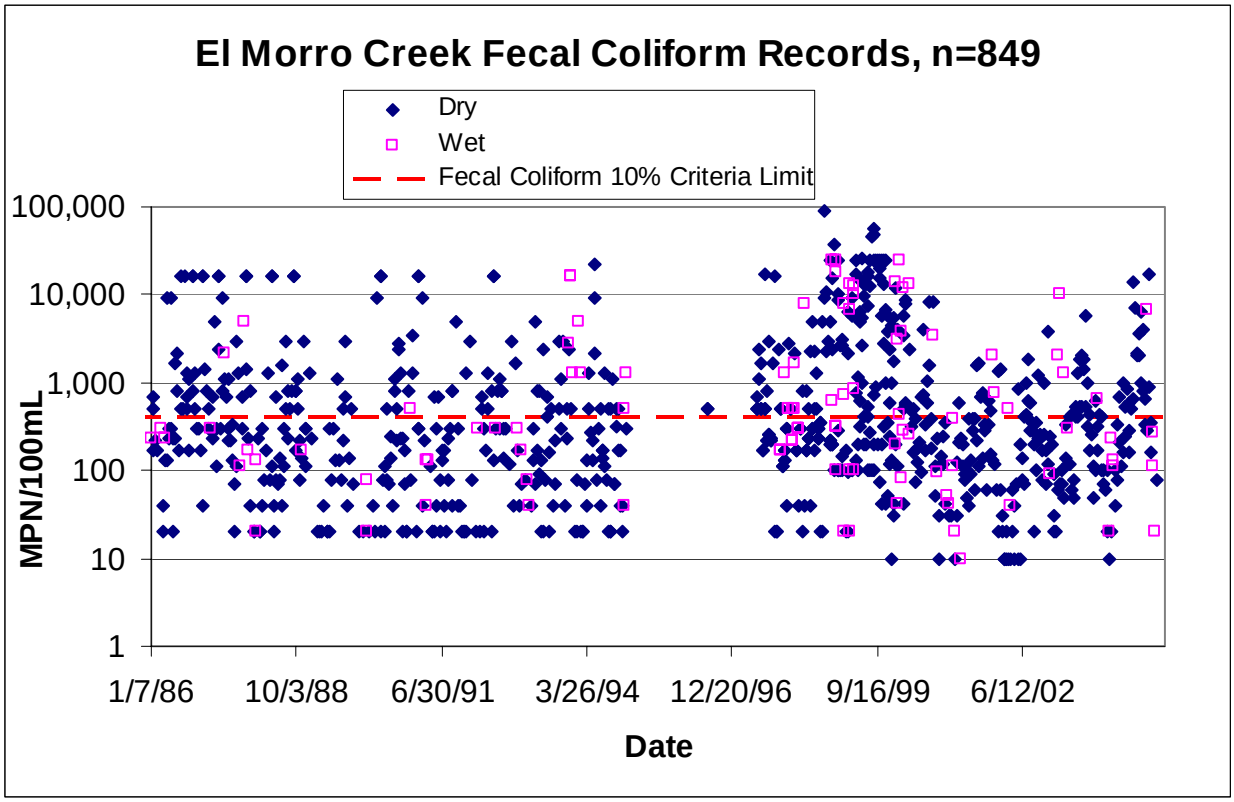


Figure A 35: El Morro Creek fecal coliform data and corresponding cumulative frequency distribution



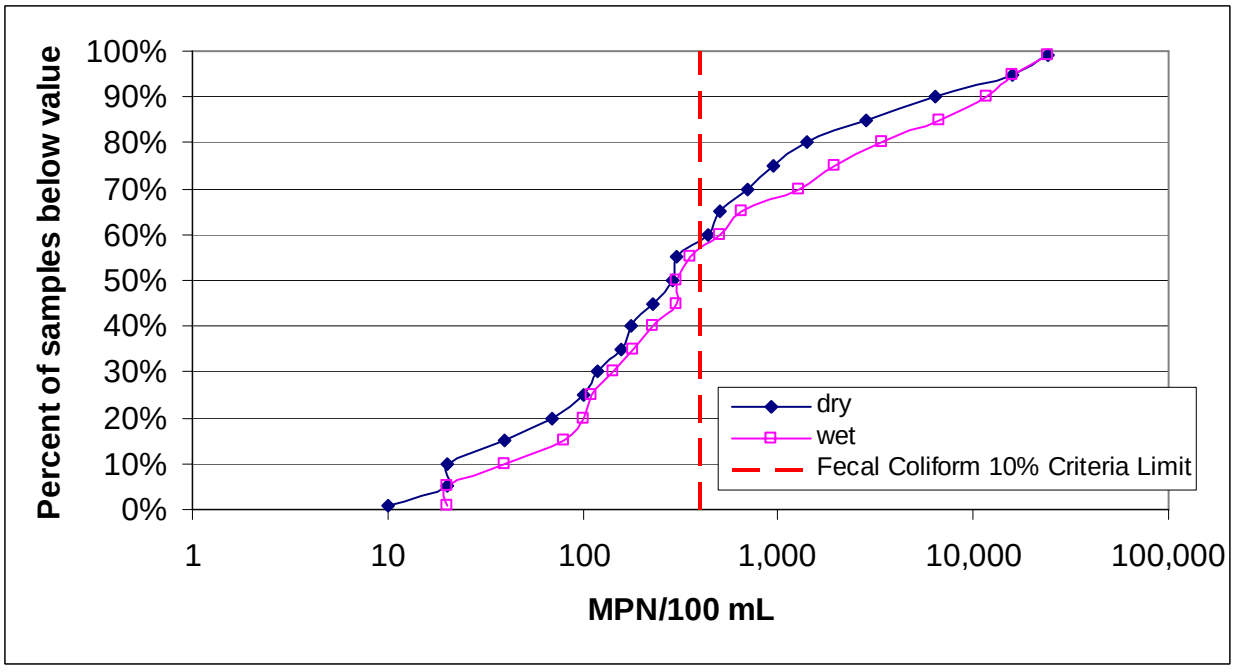
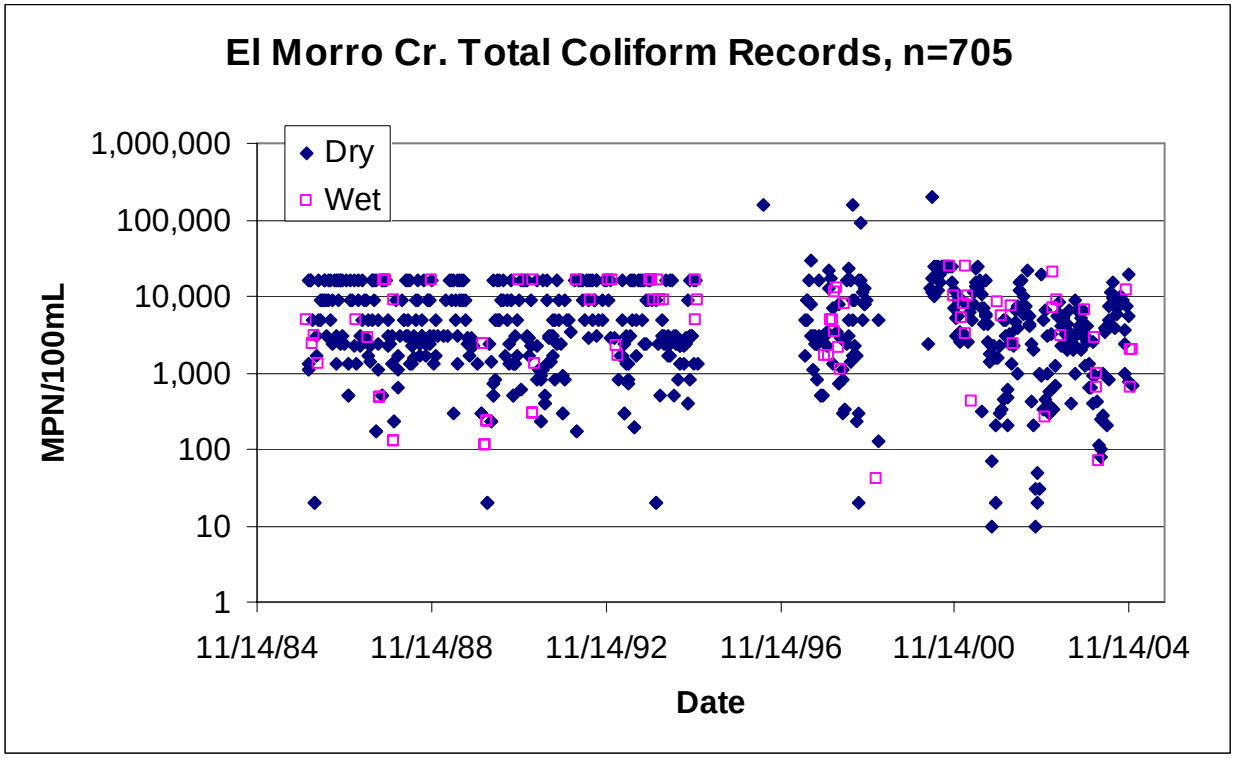


Figure A 36: El Morro Creek total coliform data and corresponding cumulative frequency distribution



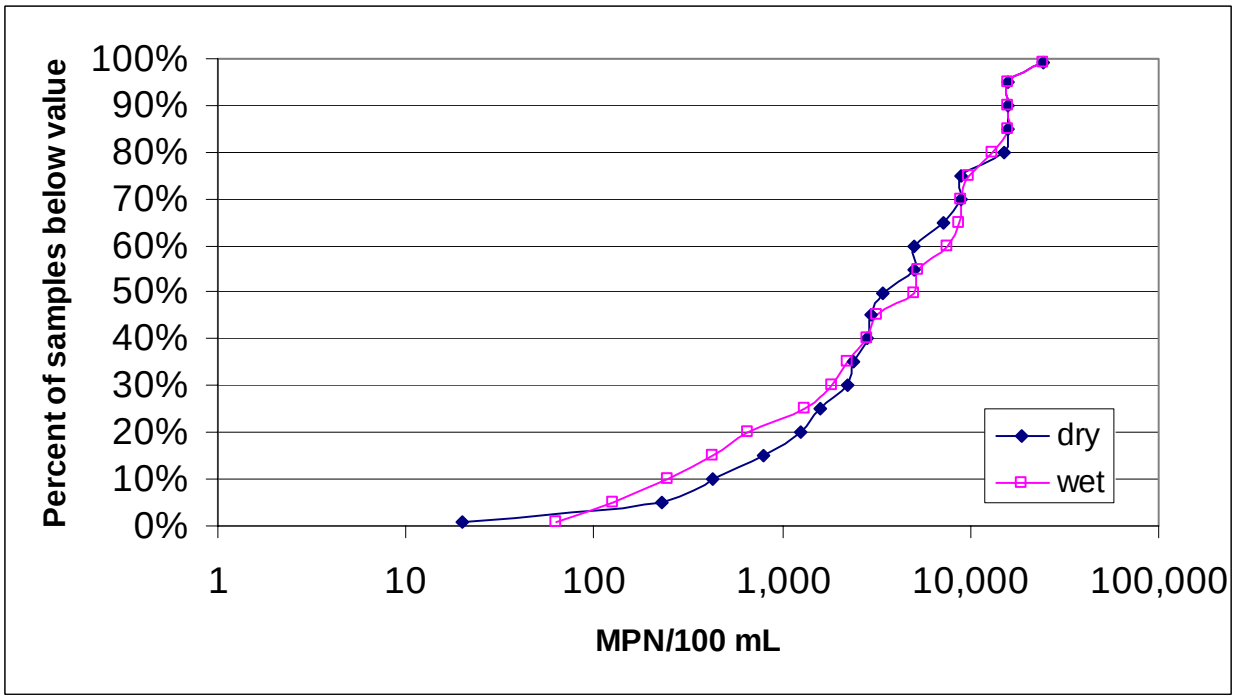


Figure A 37: Percentage of samples from El Morro Creek which exceed thresholds, by month

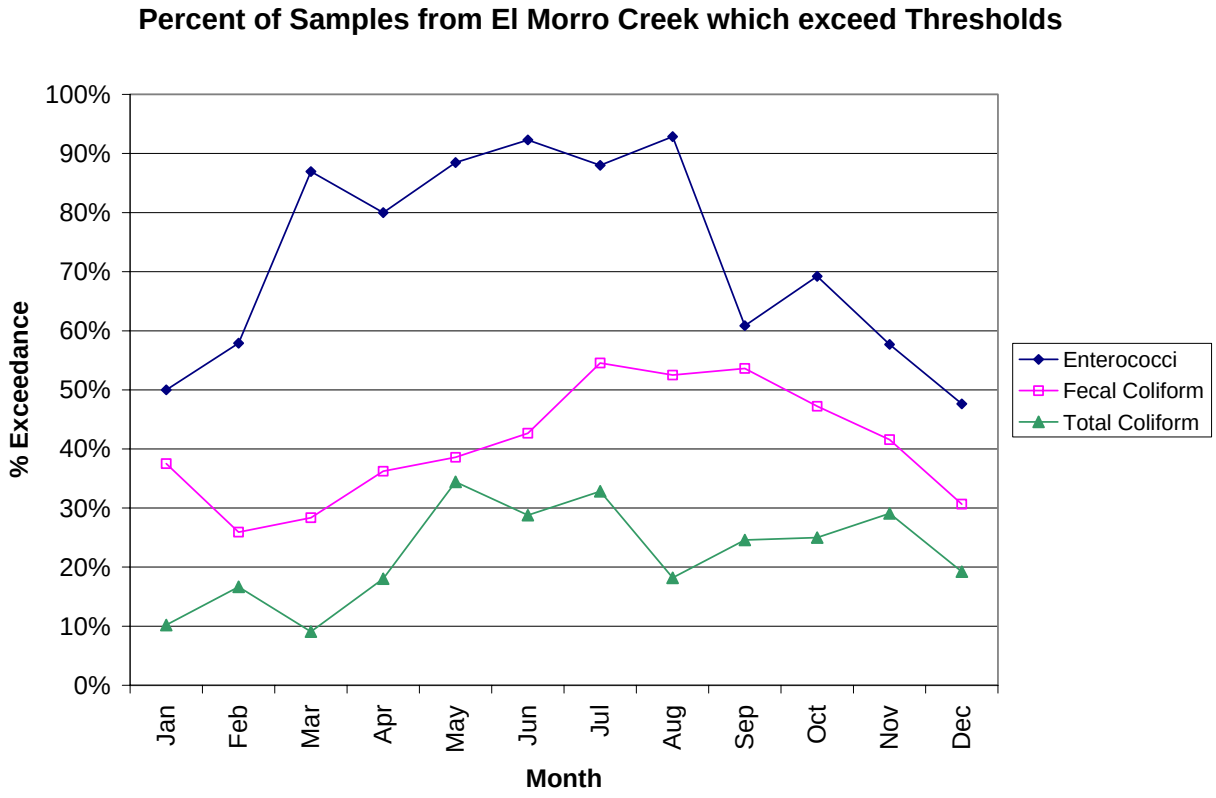
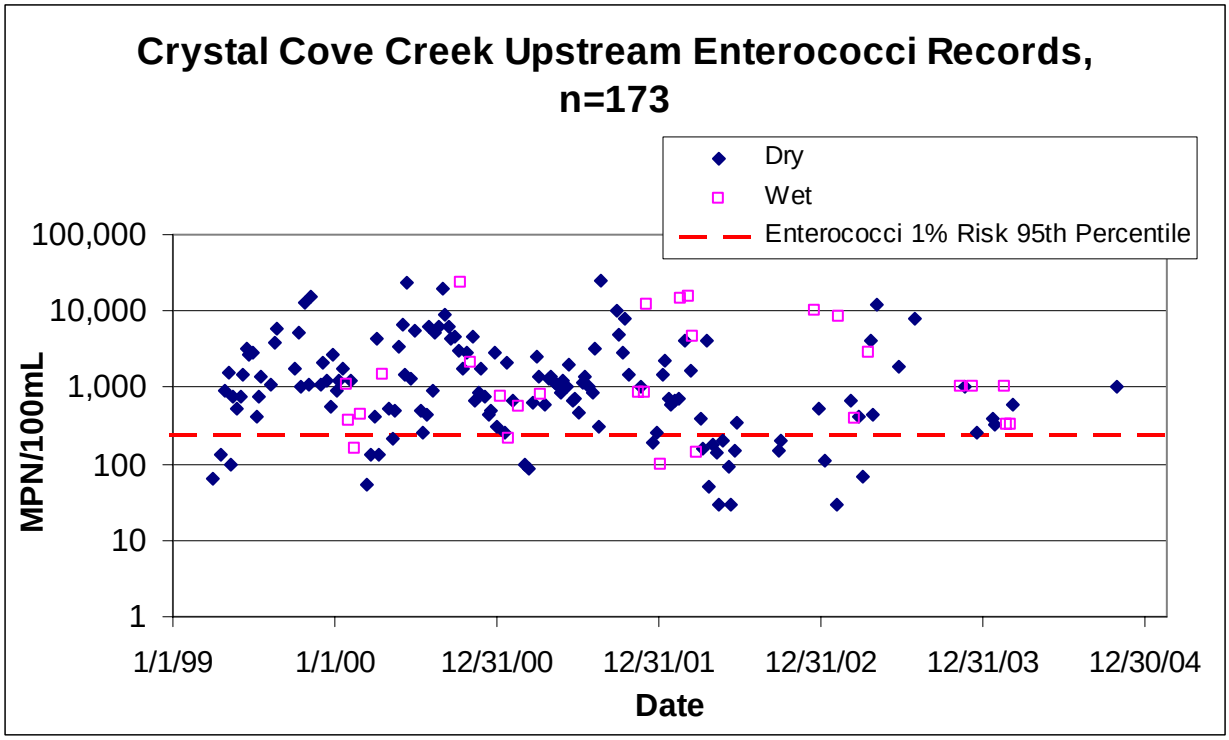


Figure A 38: Crystal Cove Creek Upstream enterococci data and corresponding cumulative frequency distribution



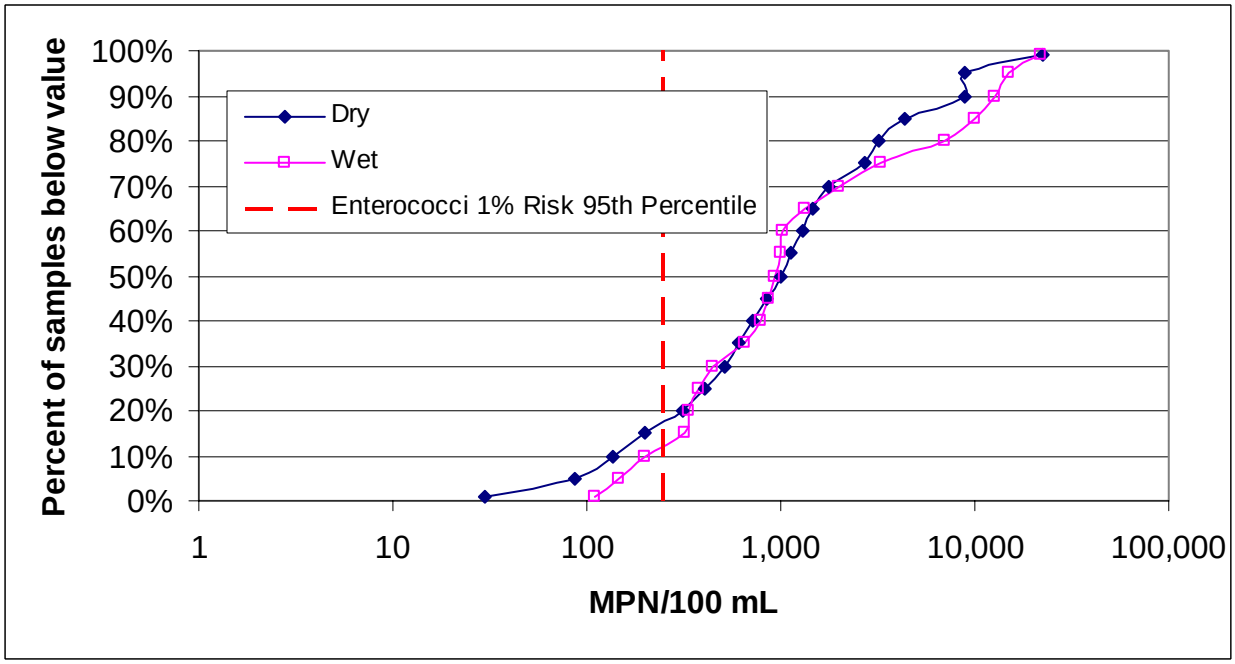
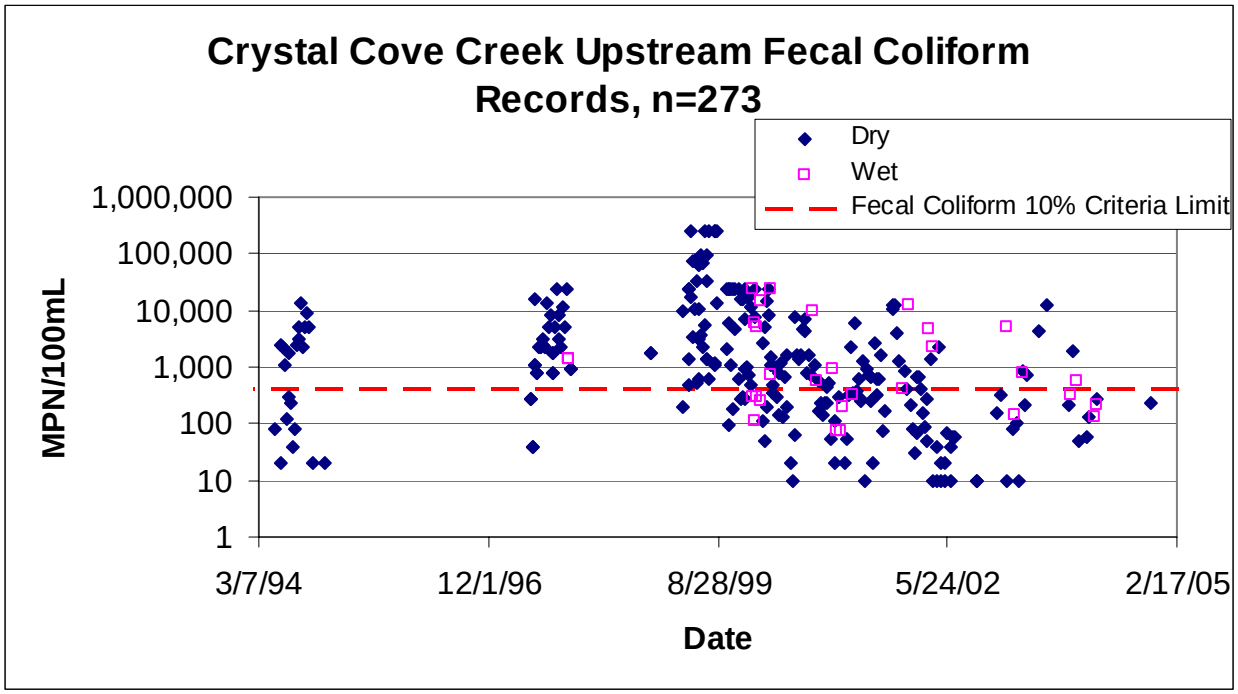


Figure A 39: Crystal Cove Creek Upstream fecal coliform data and corresponding cumulative frequency distribution



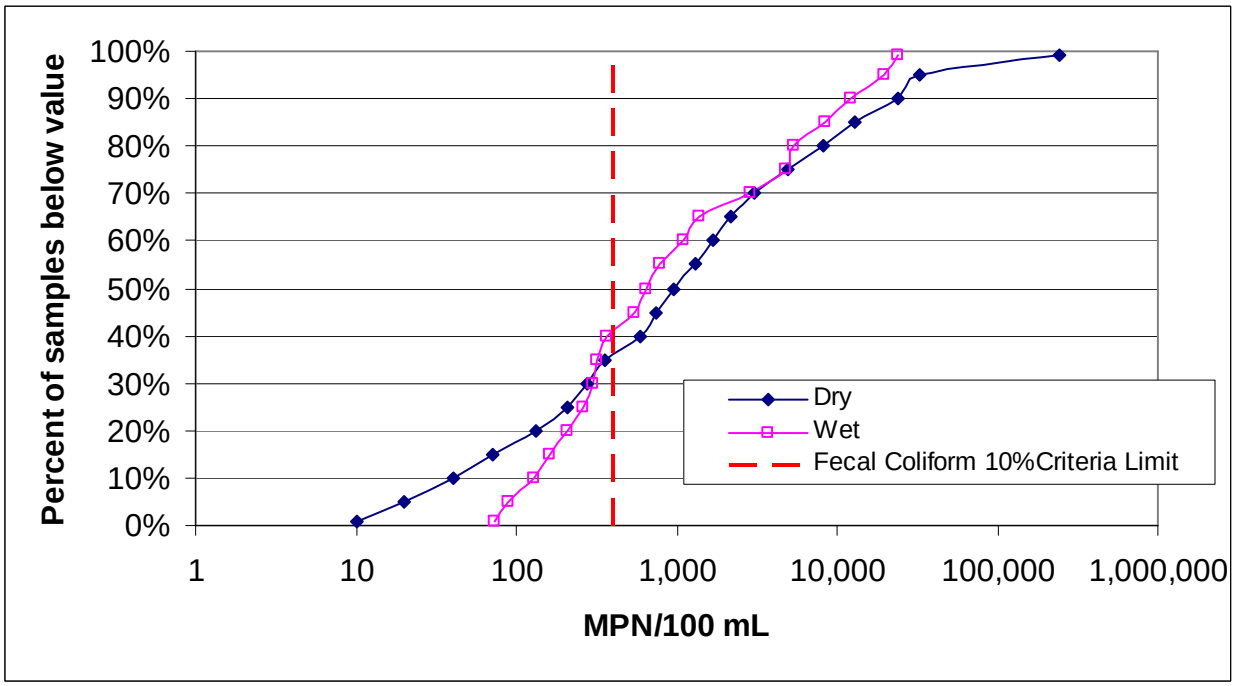
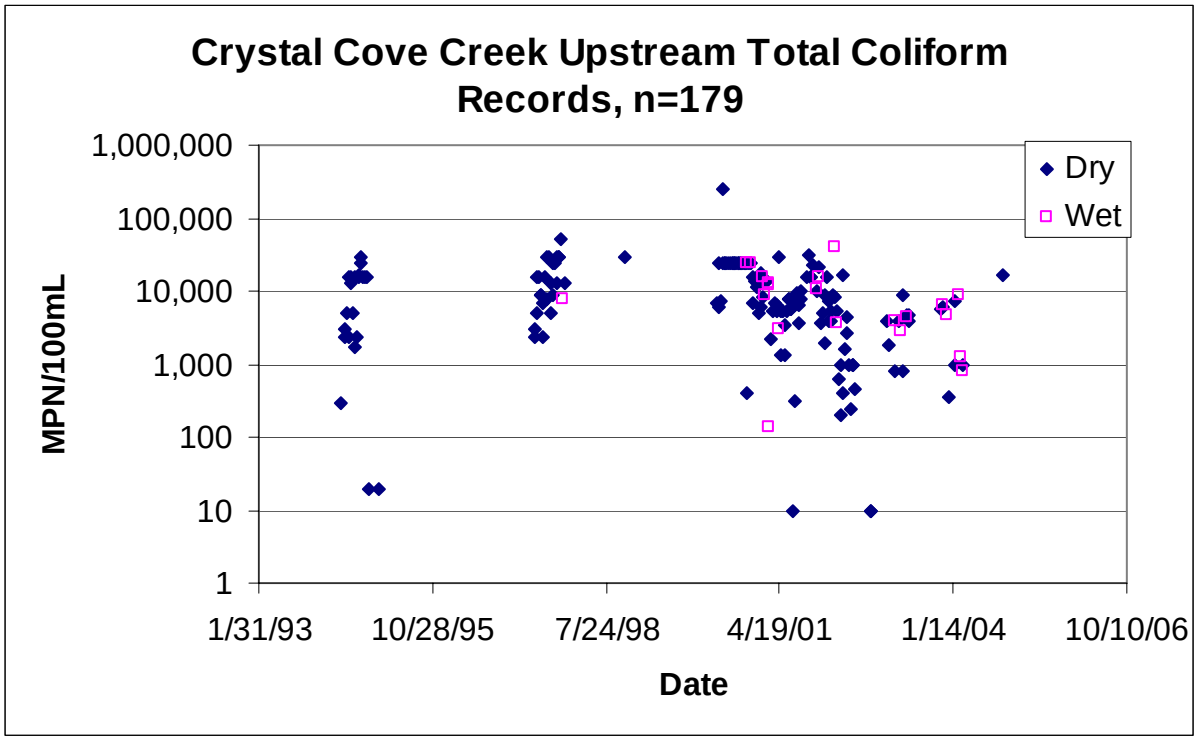


Figure A 40: Crystal Cove Creek Upstream total coliform data and corresponding cumulative frequency distribution



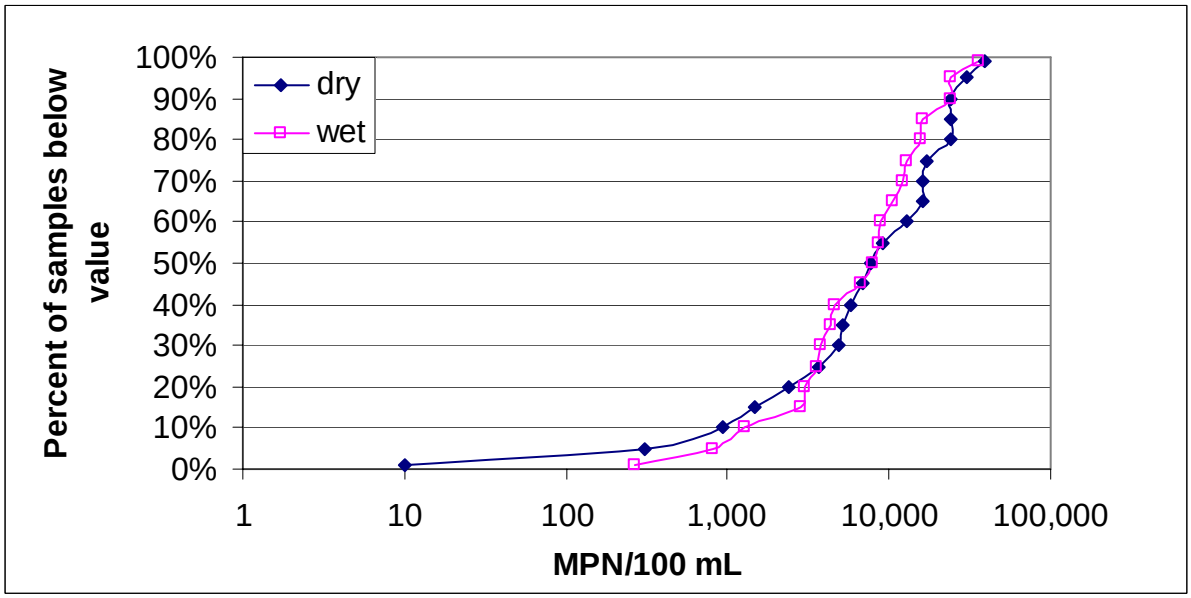


Figure A 41: Percentage of samples from Crystal Cove Creek Upstream which exceed thresholds, by month

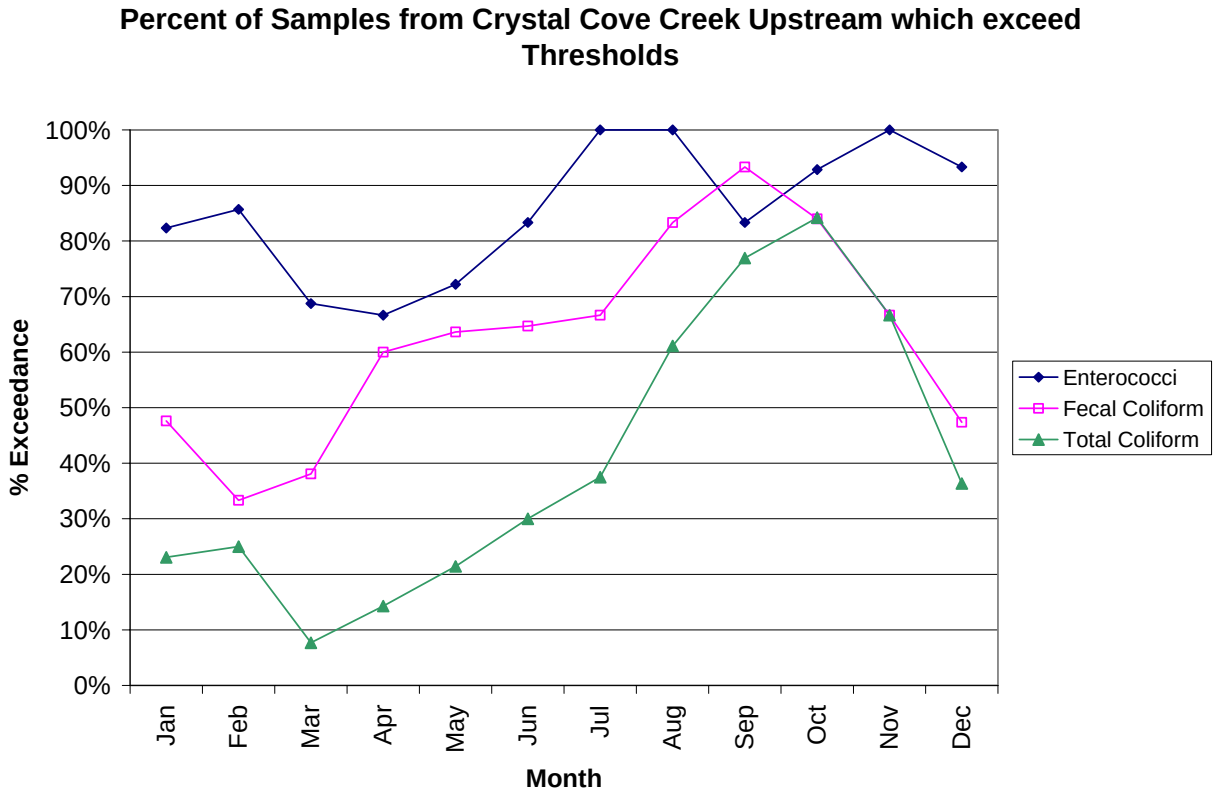
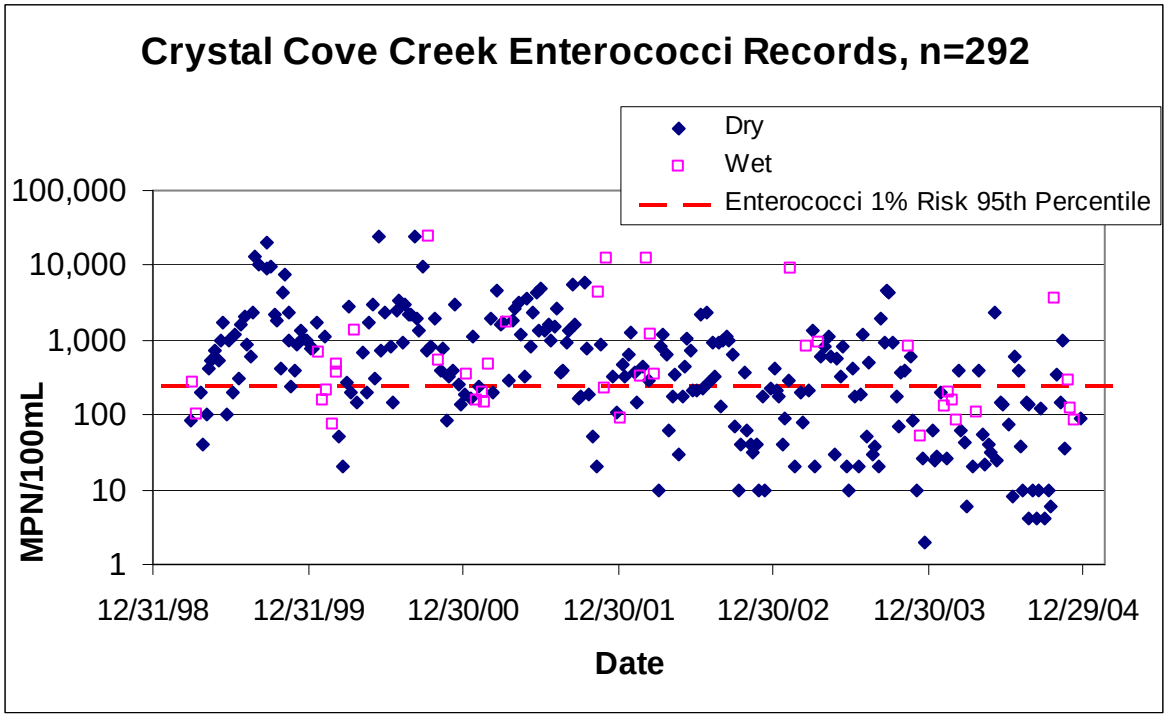


Figure A 42: Crystal Cove Creek enterococci data and corresponding cumulative frequency distribution



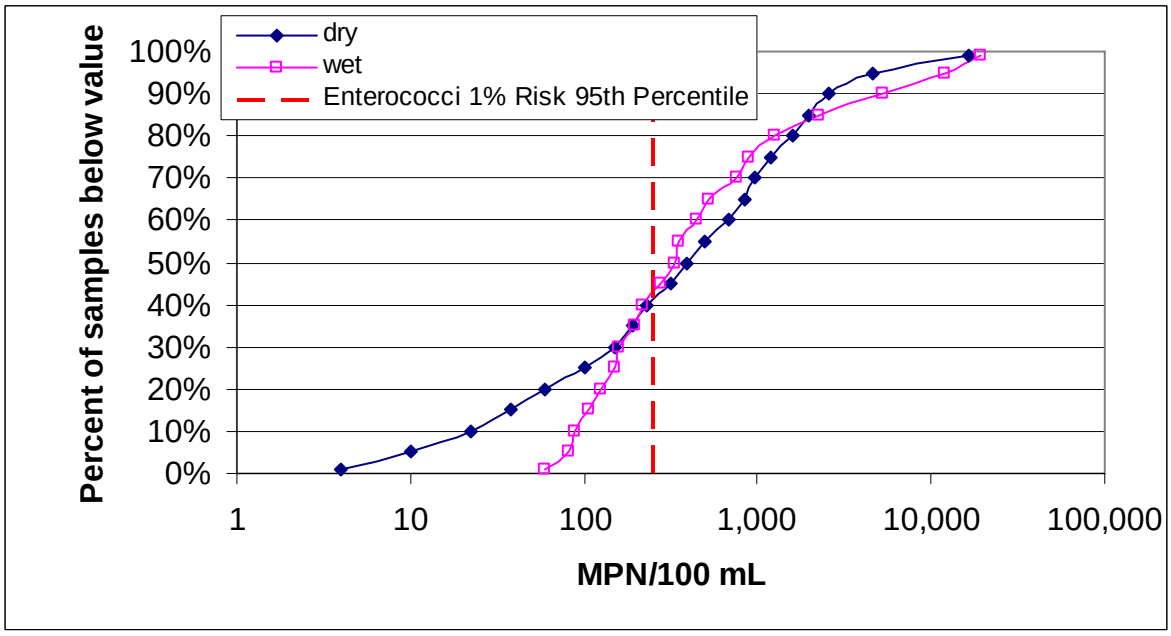
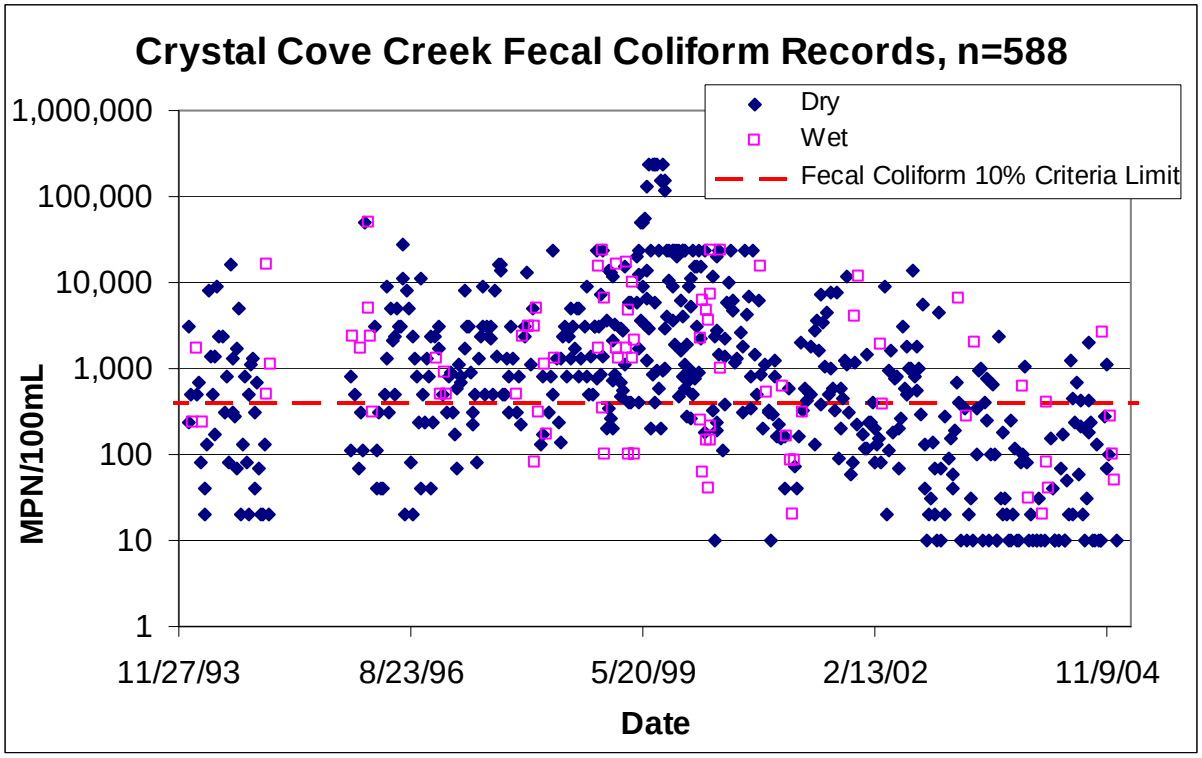


Figure A 43: Crystal Cove Creek fecal coliform data and corresponding cumulative frequency distribution



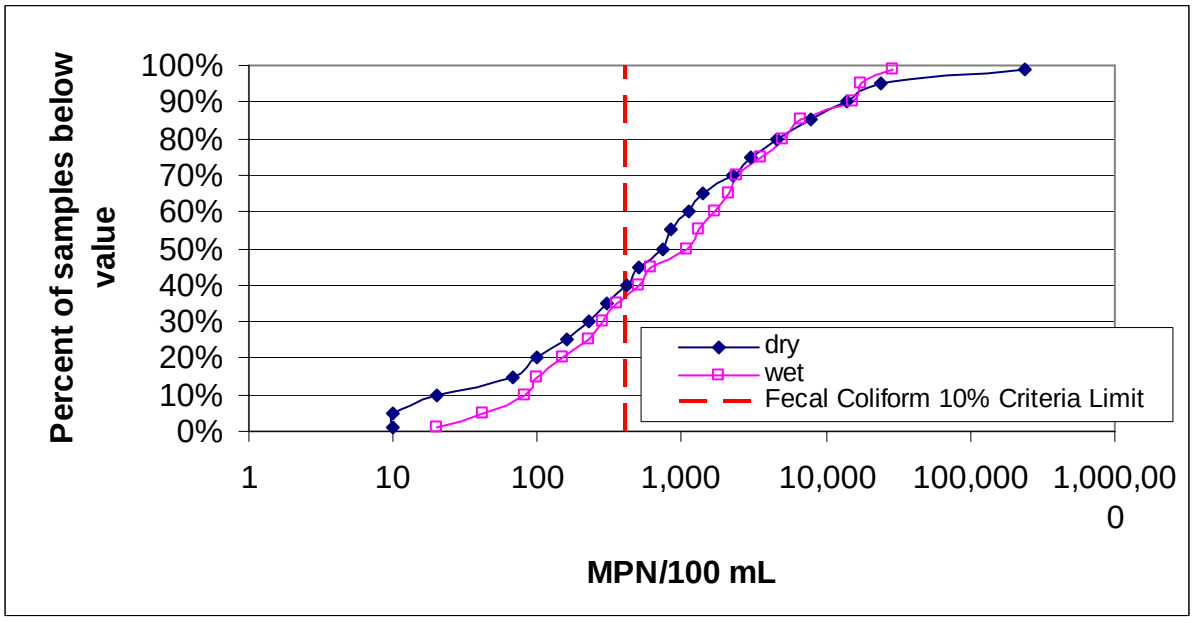
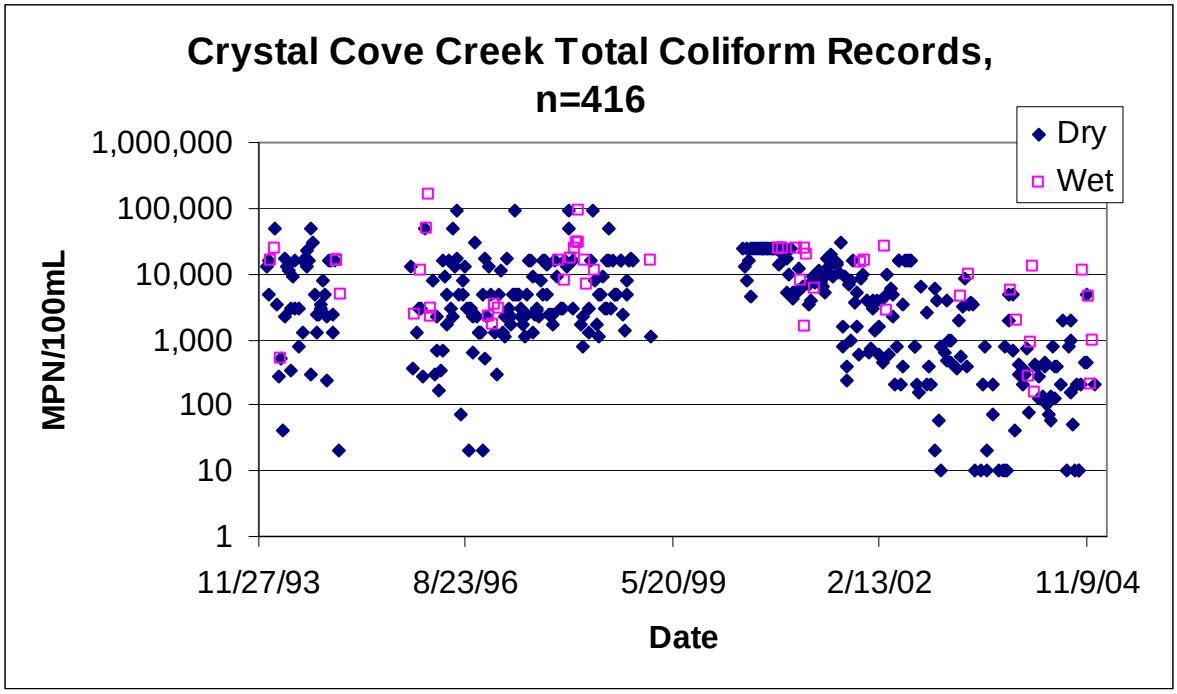


Figure A 44: Crystal Cove Creek total coliform data and corresponding cumulative frequency distribution



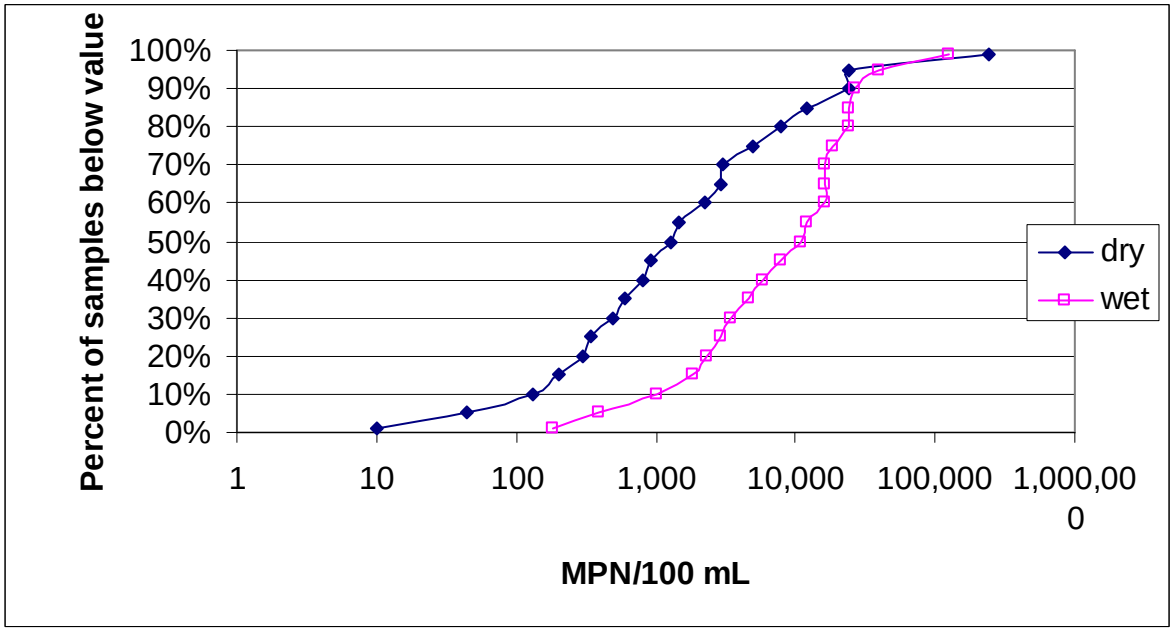


Figure A 45: Percentage of samples from Crystal Cove Creek which exceed thresholds, by month

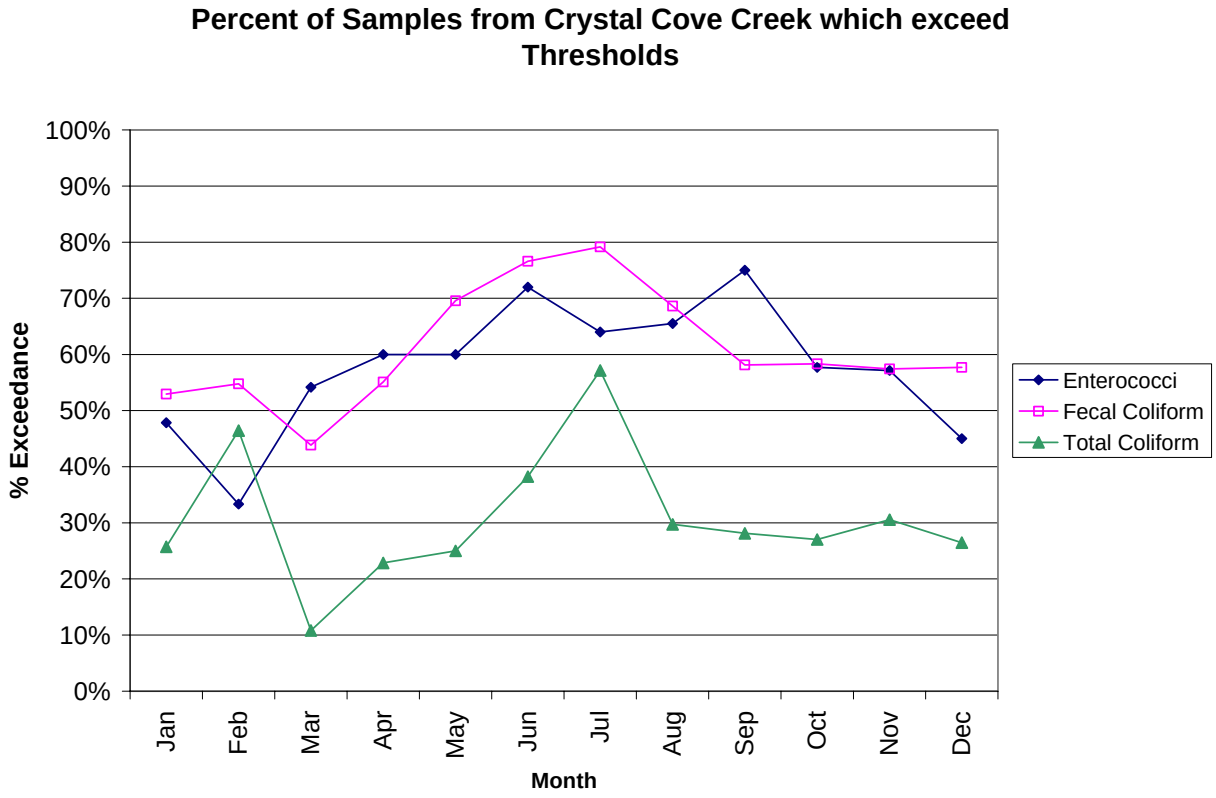
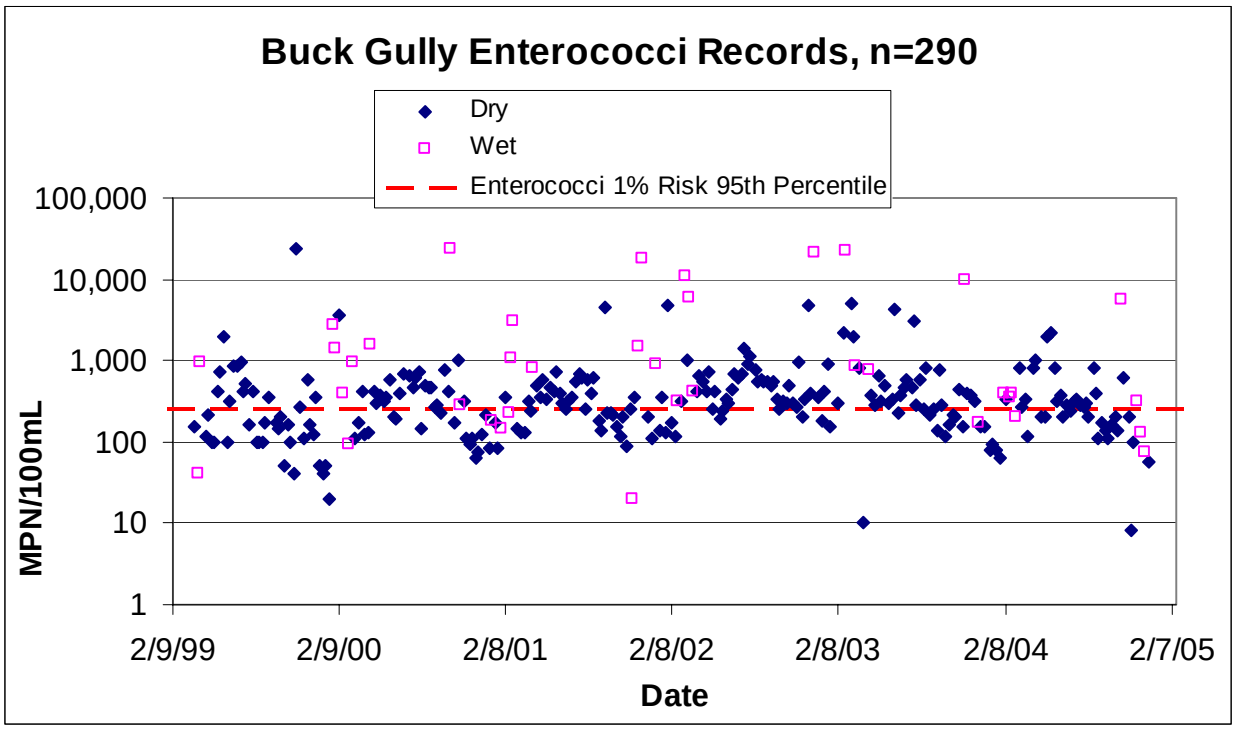


Figure A 46: Buck Gully enterococci data and corresponding cumulative frequency distribution



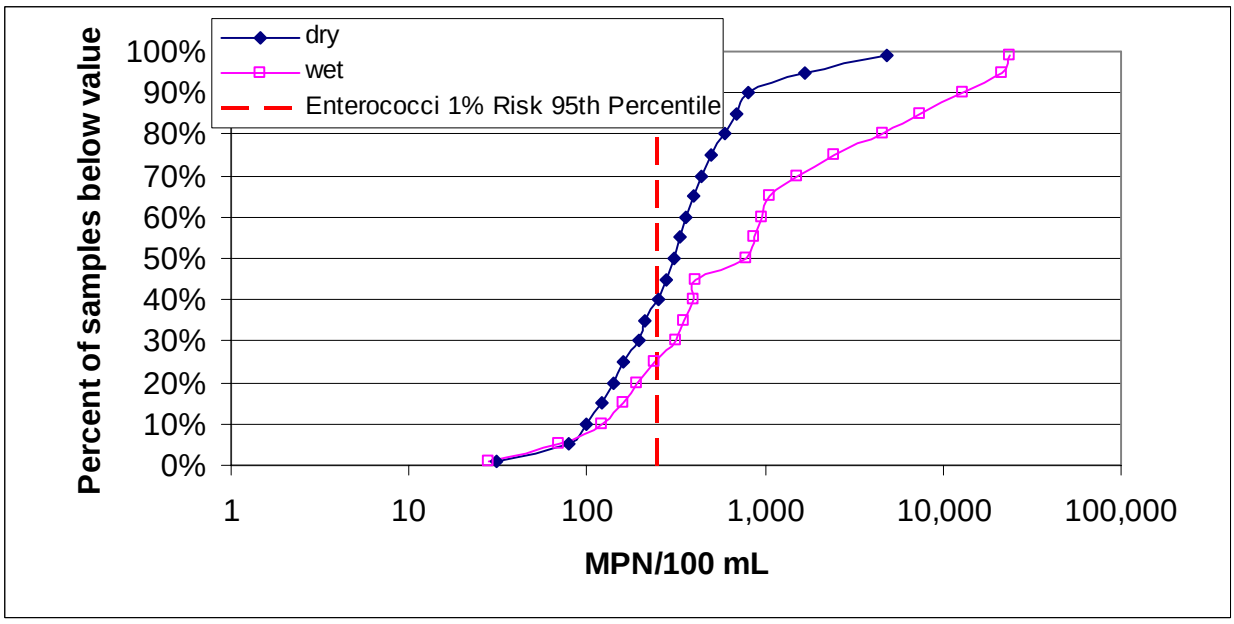
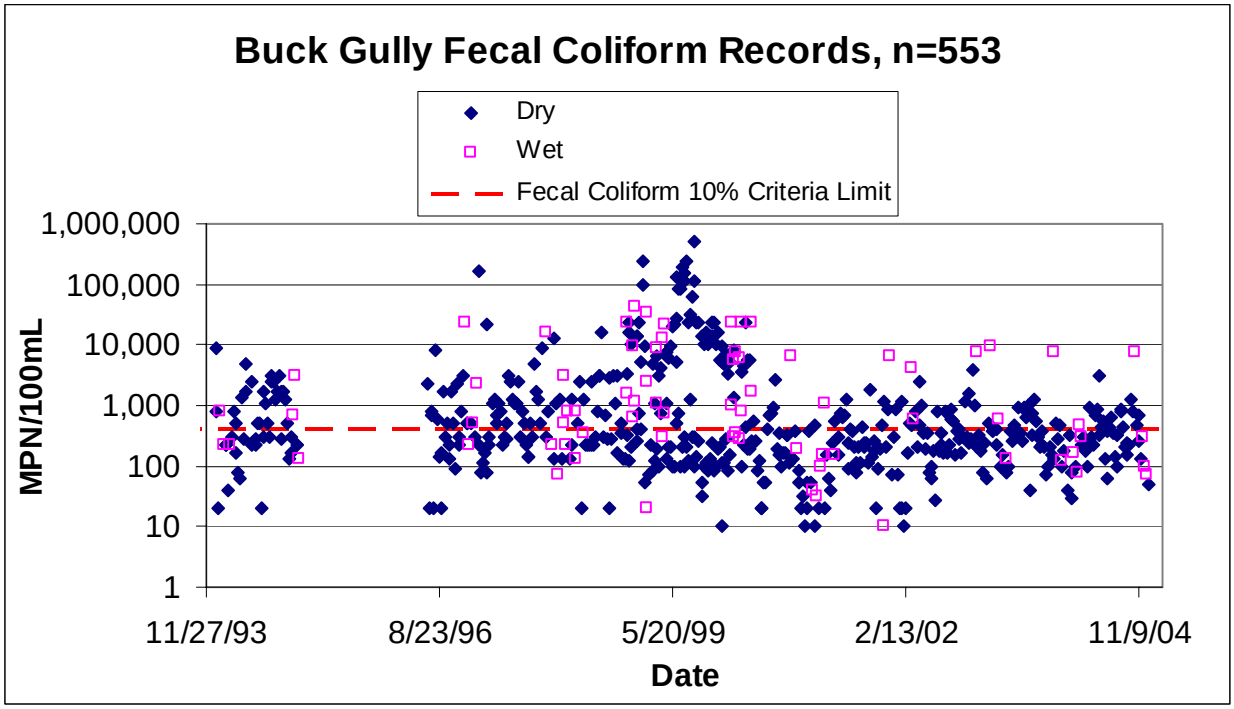


Figure A 47: Buck Gully fecal coliform data and corresponding cumulative frequency distribution



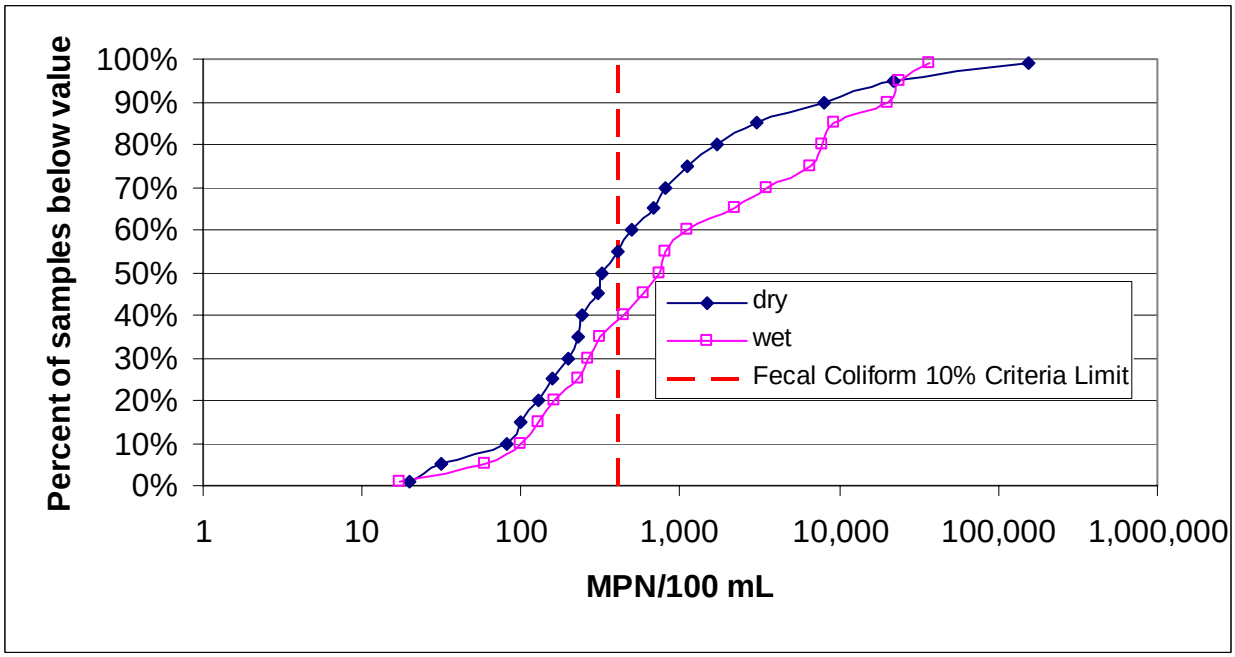
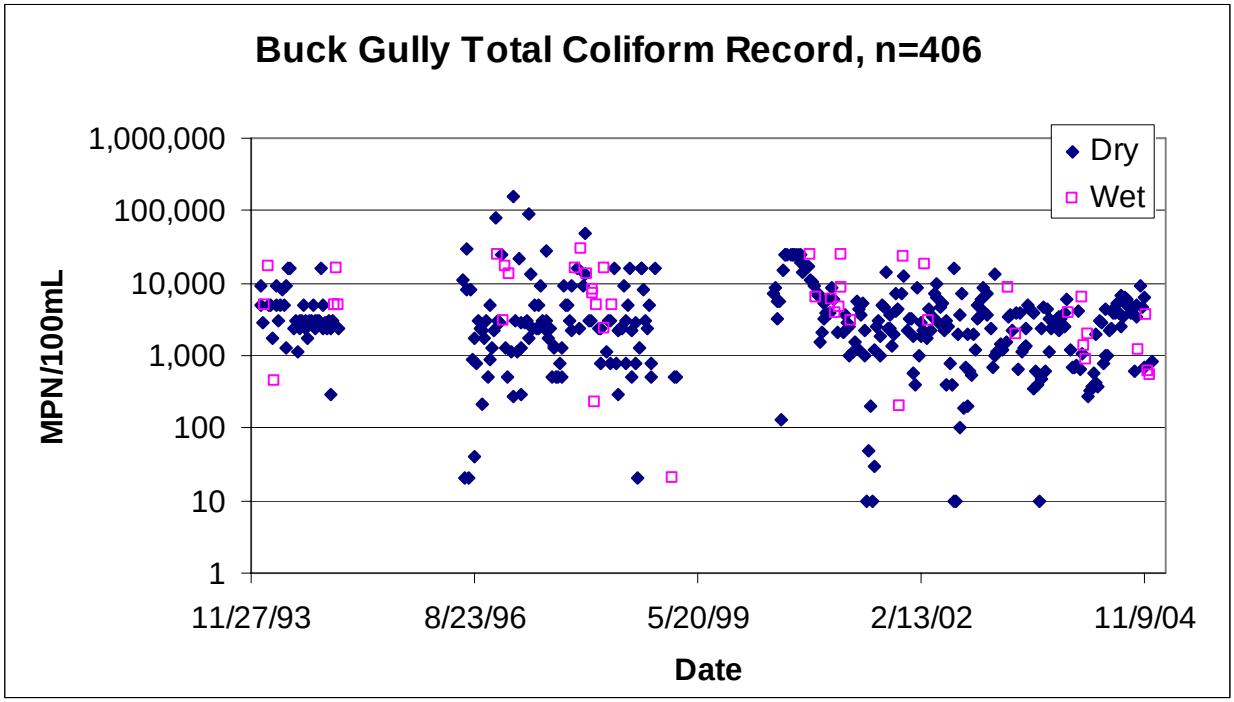


Figure A 48: Buck Gully total coliform data and corresponding cumulative frequency distribution



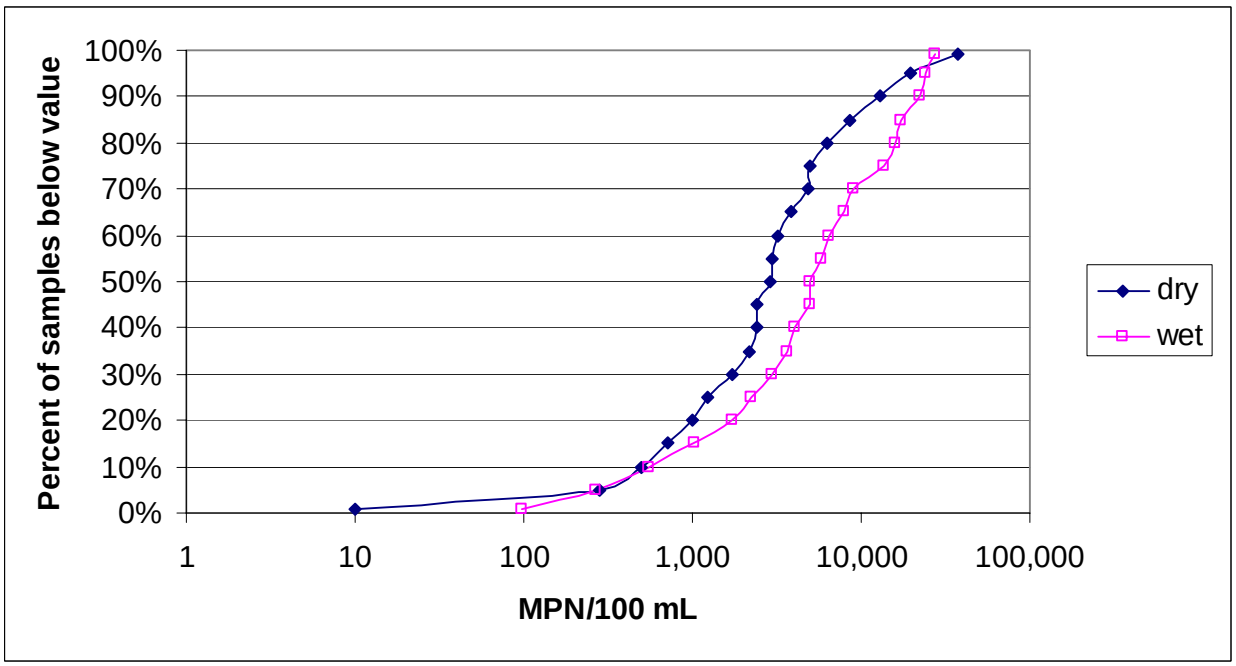


Figure A 49: Percentage of samples from Buck Gully which exceed thresholds, by month

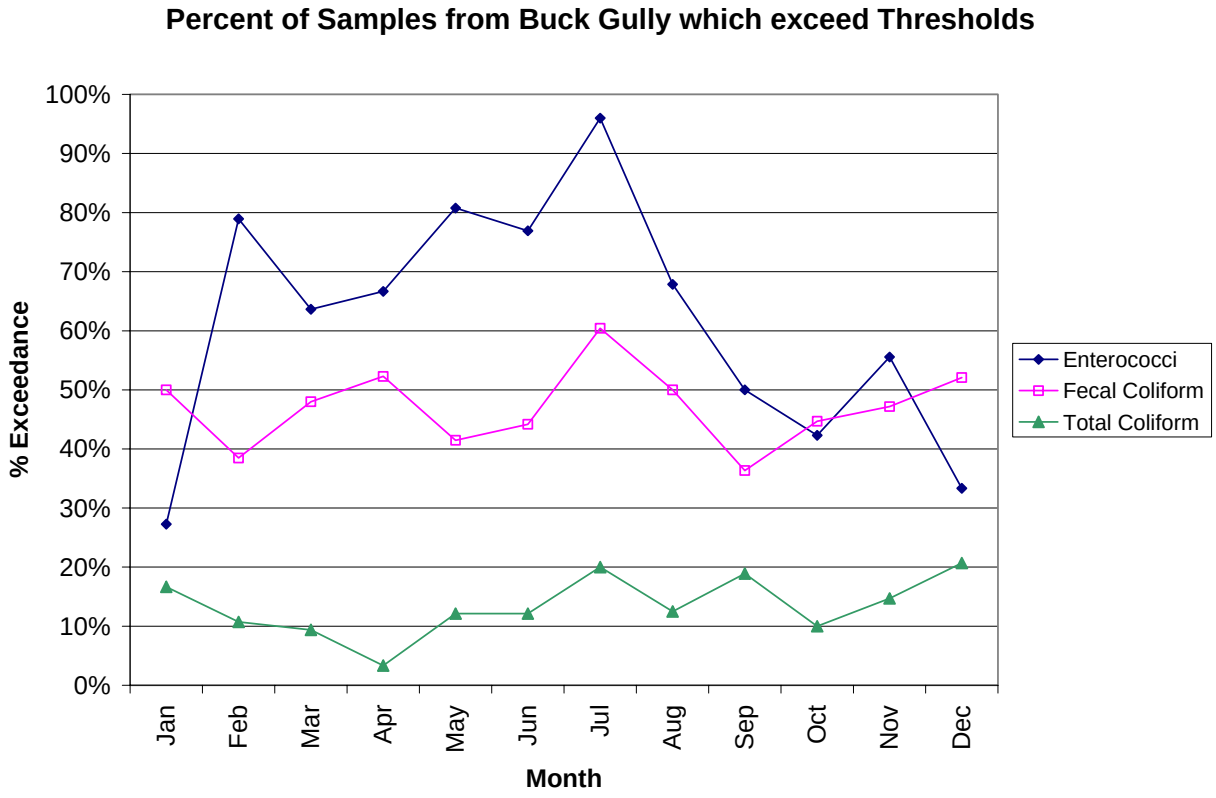
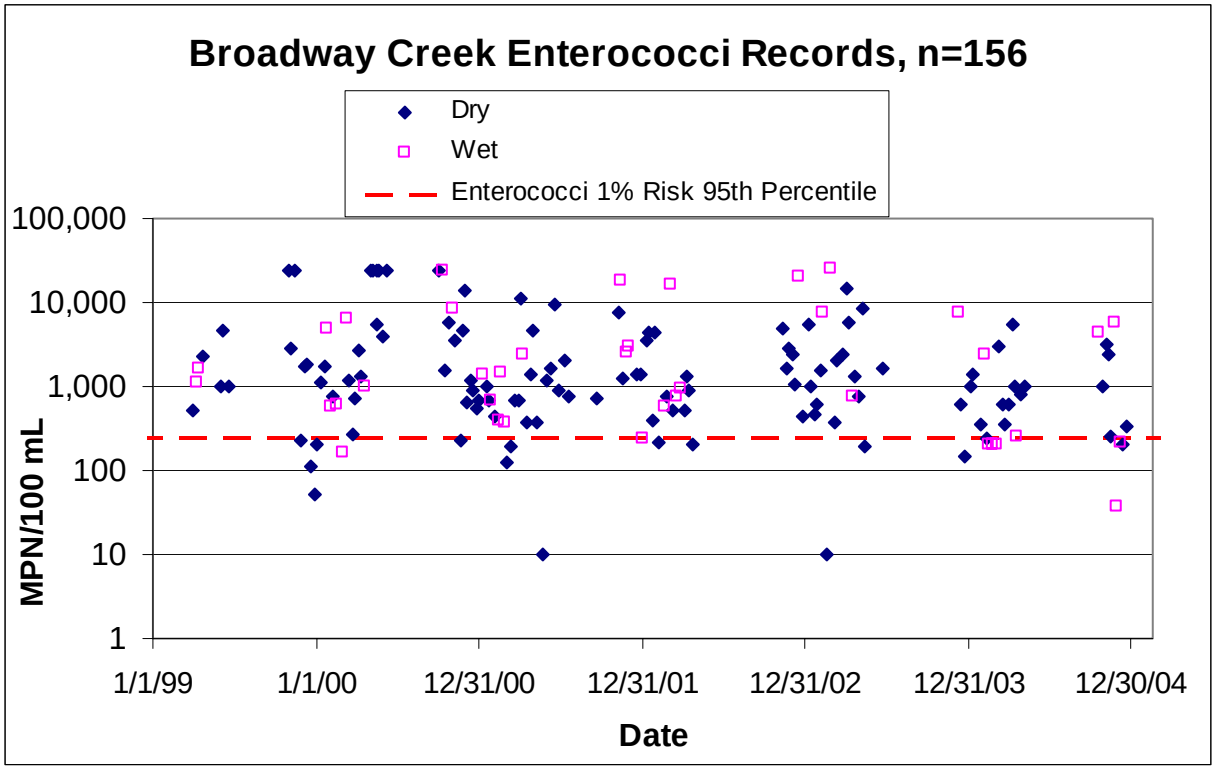


Figure A 50: Broadway Creek enterococci data and corresponding cumulative frequency distribution



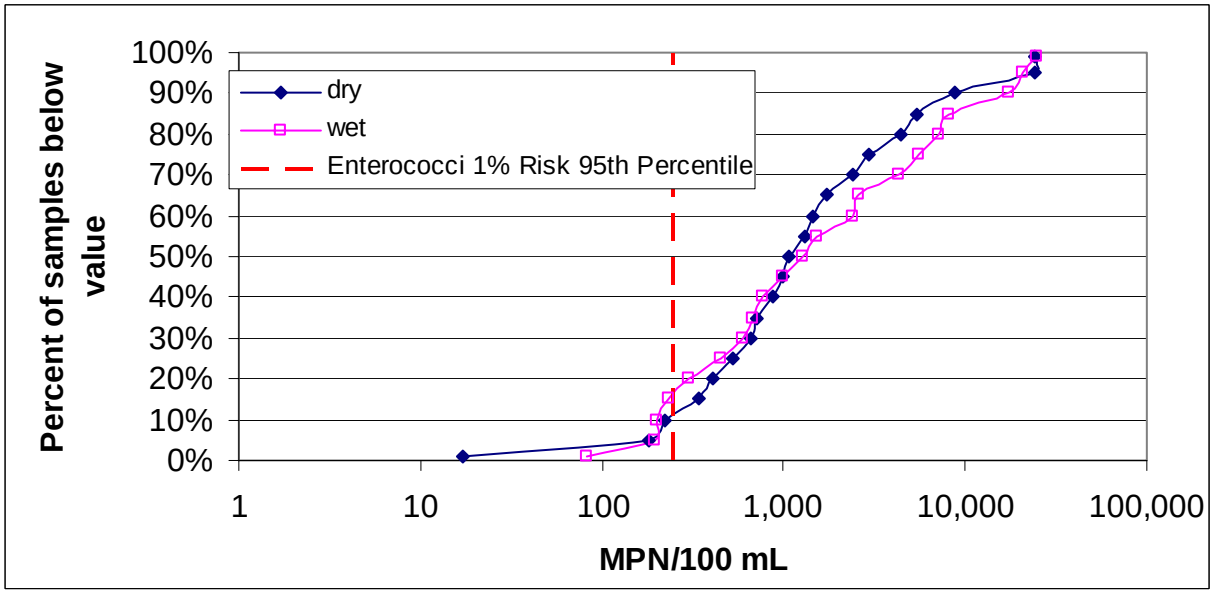
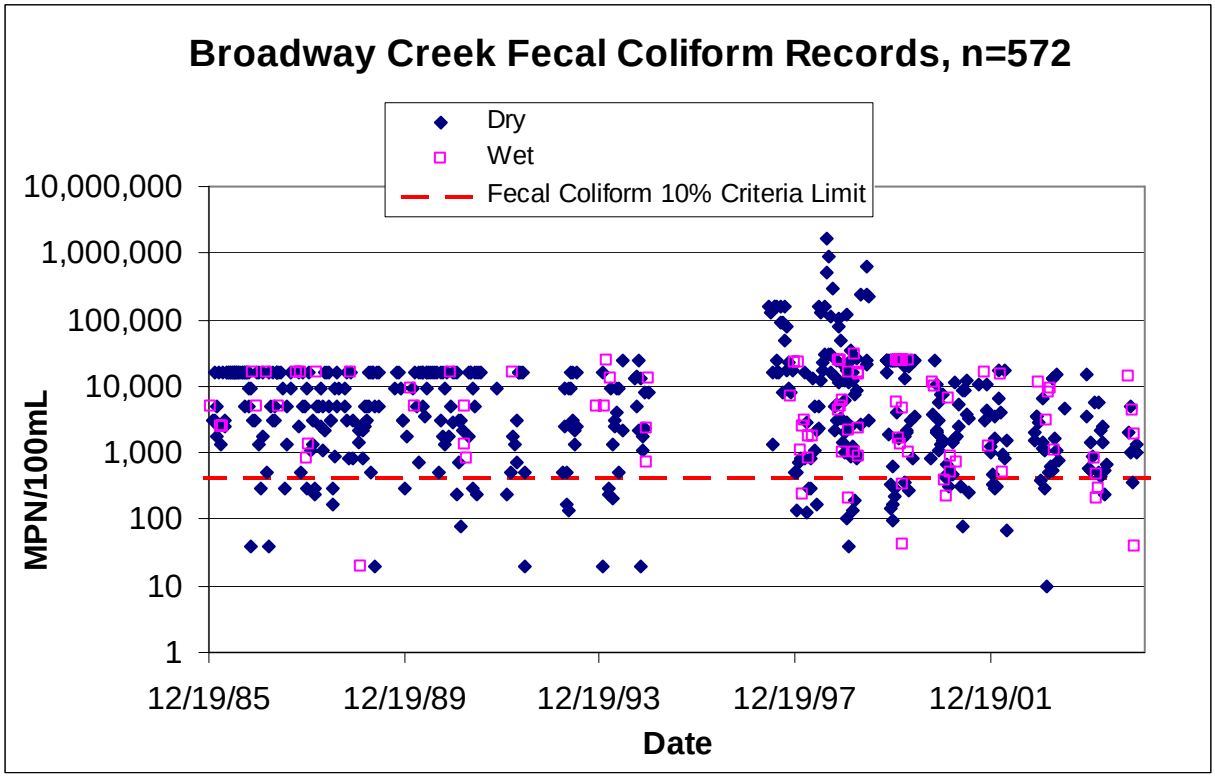


Figure A 51: Broadway Creek fecal coliform data and corresponding cumulative frequency distribution



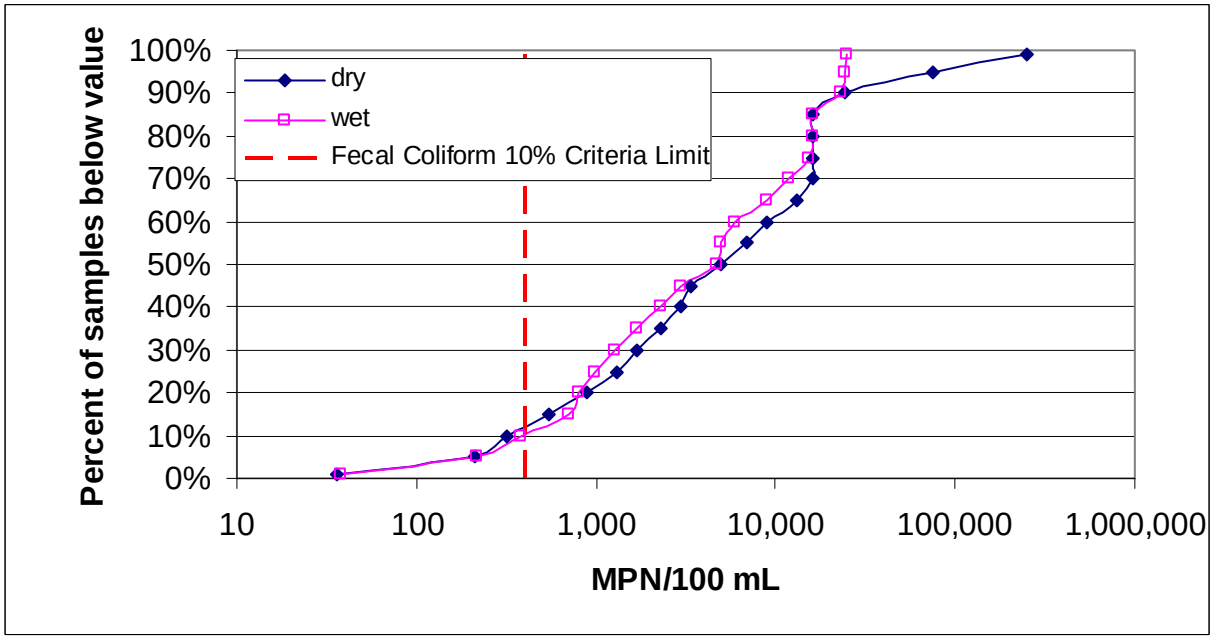
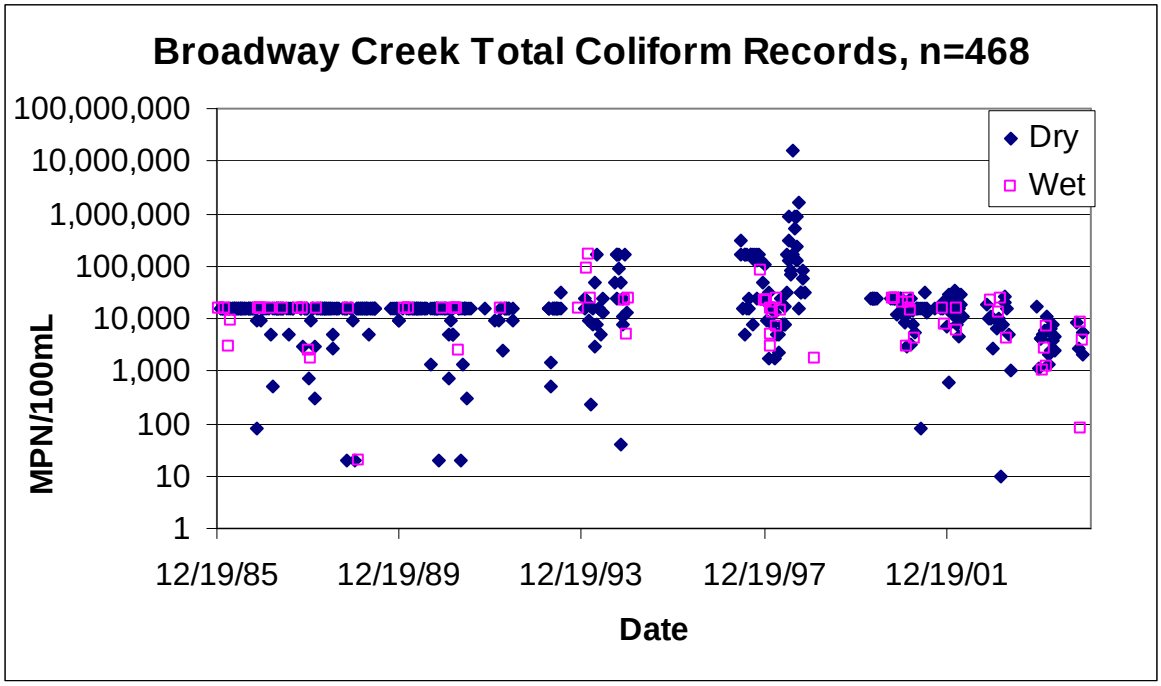


Figure A 52: Broadway Creek total coliform data and corresponding cumulative frequency distribution



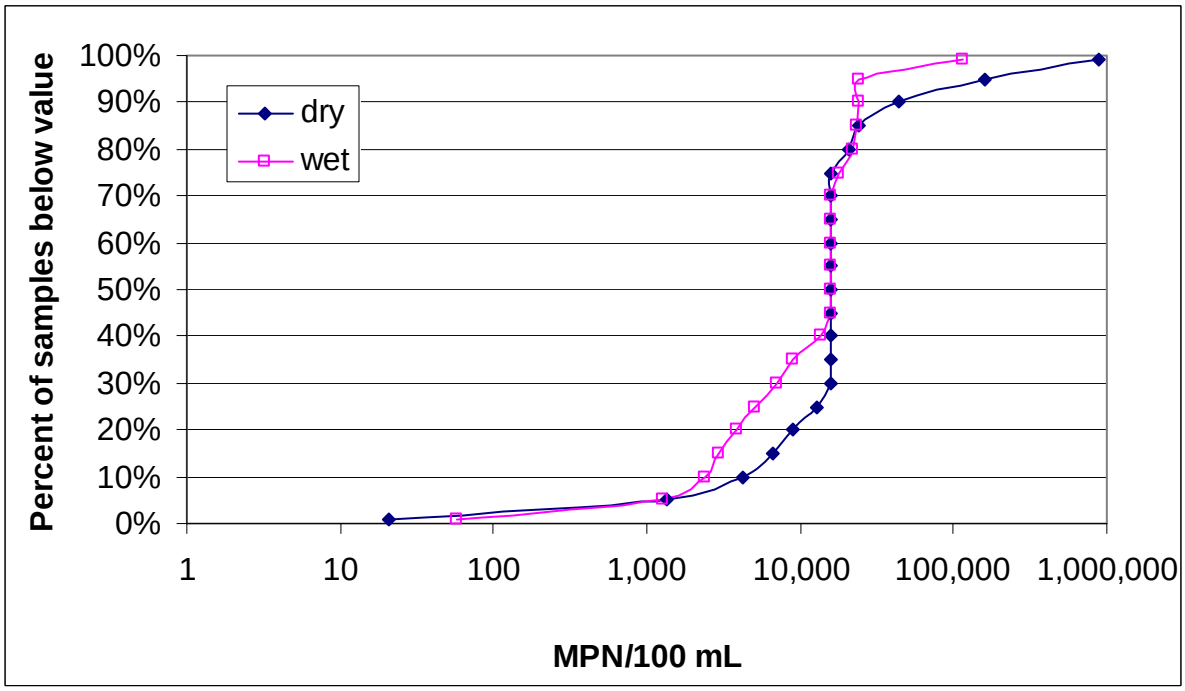
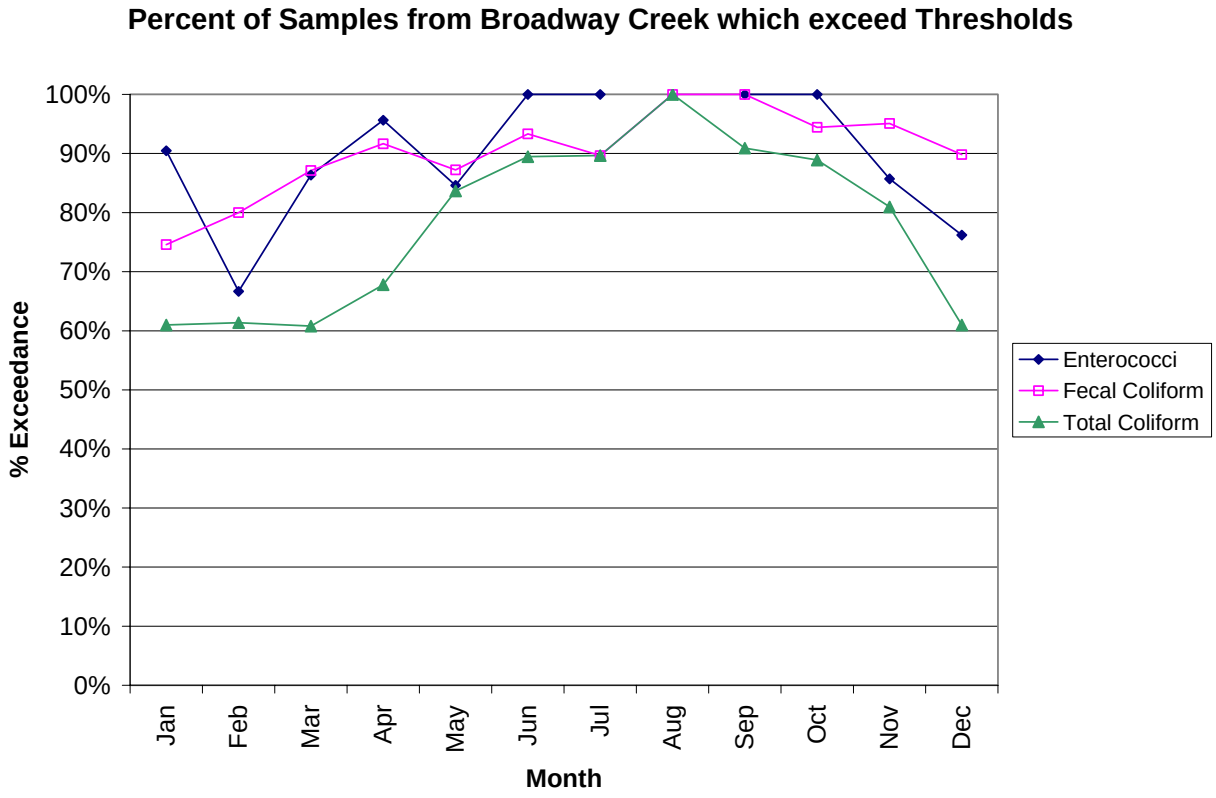


Figure A 53: Percentage of samples from Broadway Creek which exceed thresholds, by month



APPENDIX B

DATA FROM SANTA ANA REGION

FIGURES REPRODUCED FROM CDM 2005

Figure B 1: Santa Ana Watershed and sites selected by CDM for detailed bacteriological analysis (CDM 2005 Figure 19)

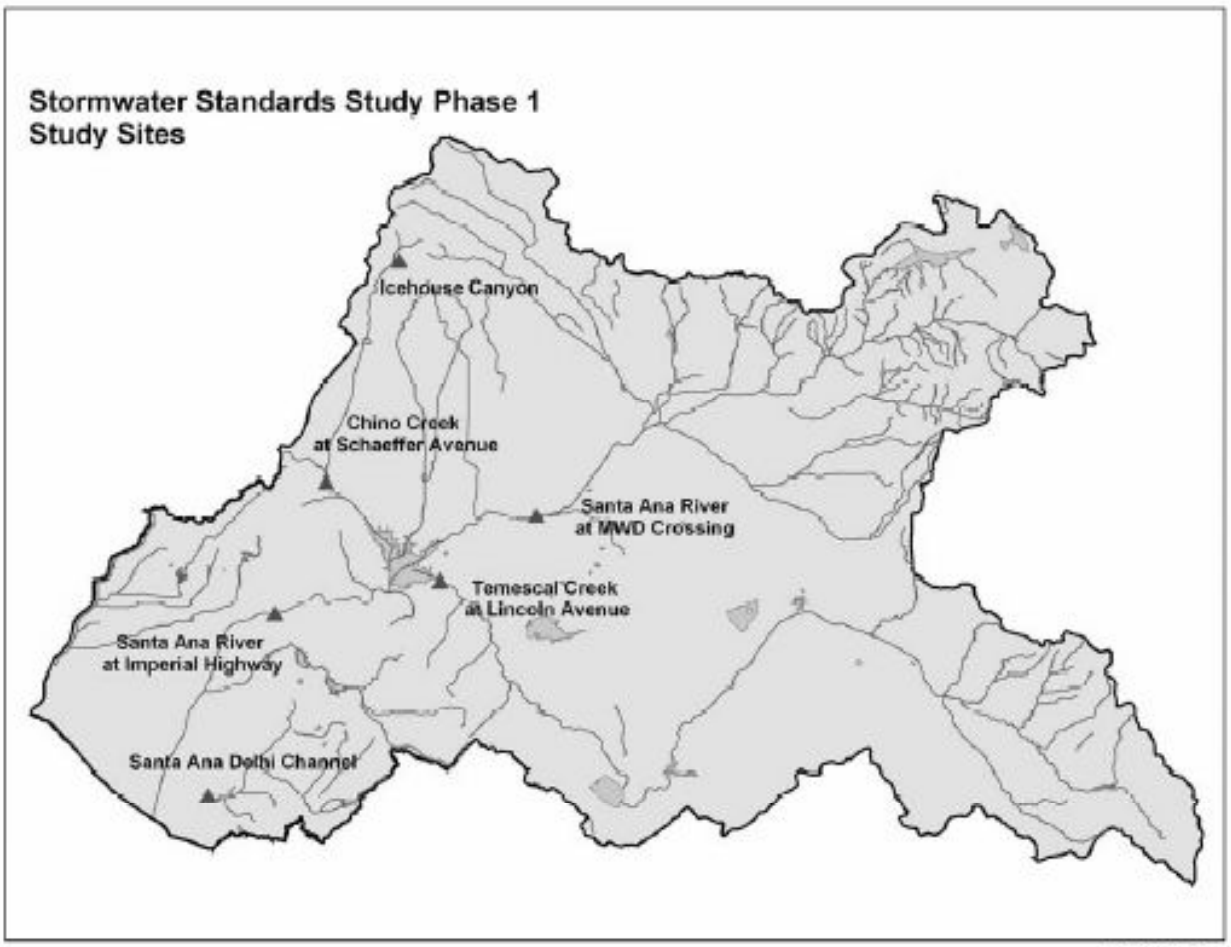


Figure 19
Study Sites Selected for Detailed Analysis

Figure B 2: Flow rate and bacteria concentration, Chino Creek (CDM 2005 Figure 35)

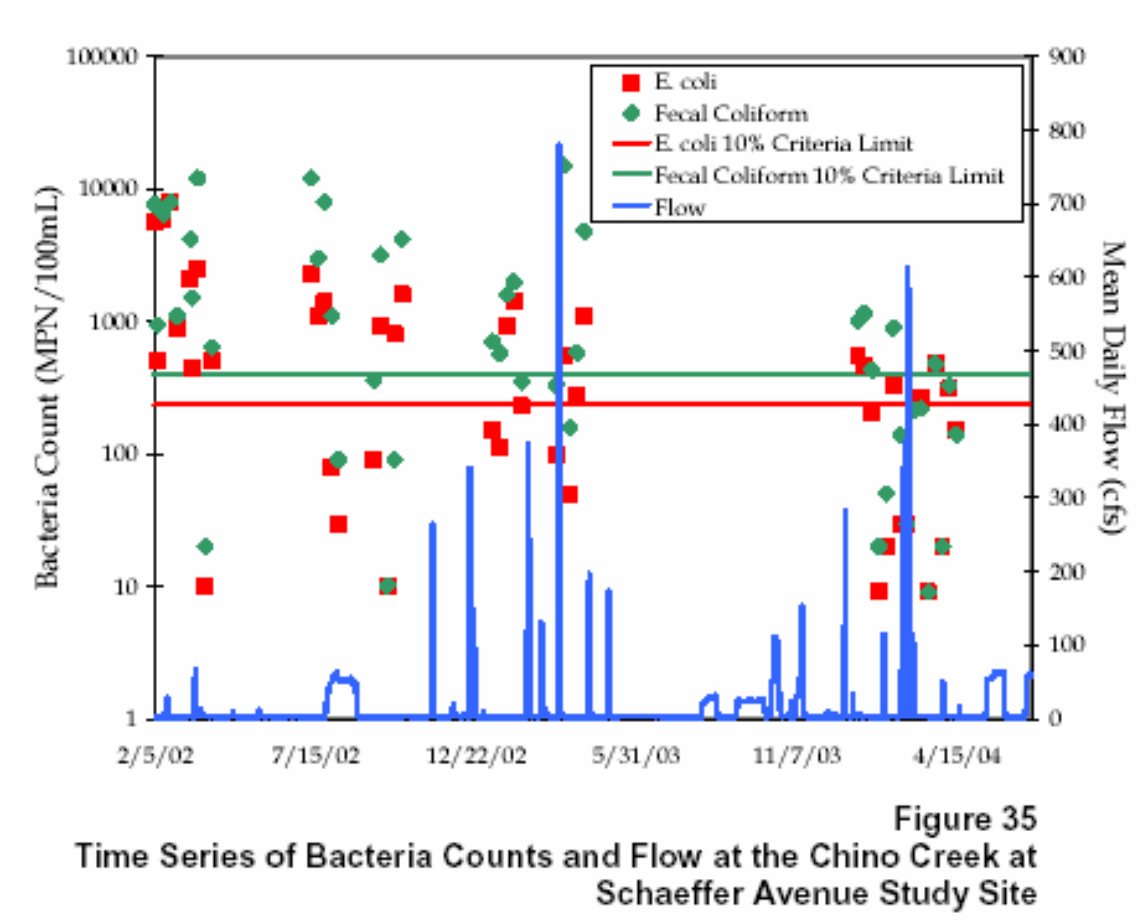


Figure 35
Time Series of Bacteria Counts and Flow at the Chino Creek at Schaeffer Avenue Study Site

Figure B 3: Flow rate and bacteria concentration, Santa Ana Delhi Channel (CDM 2005 Figure 53)

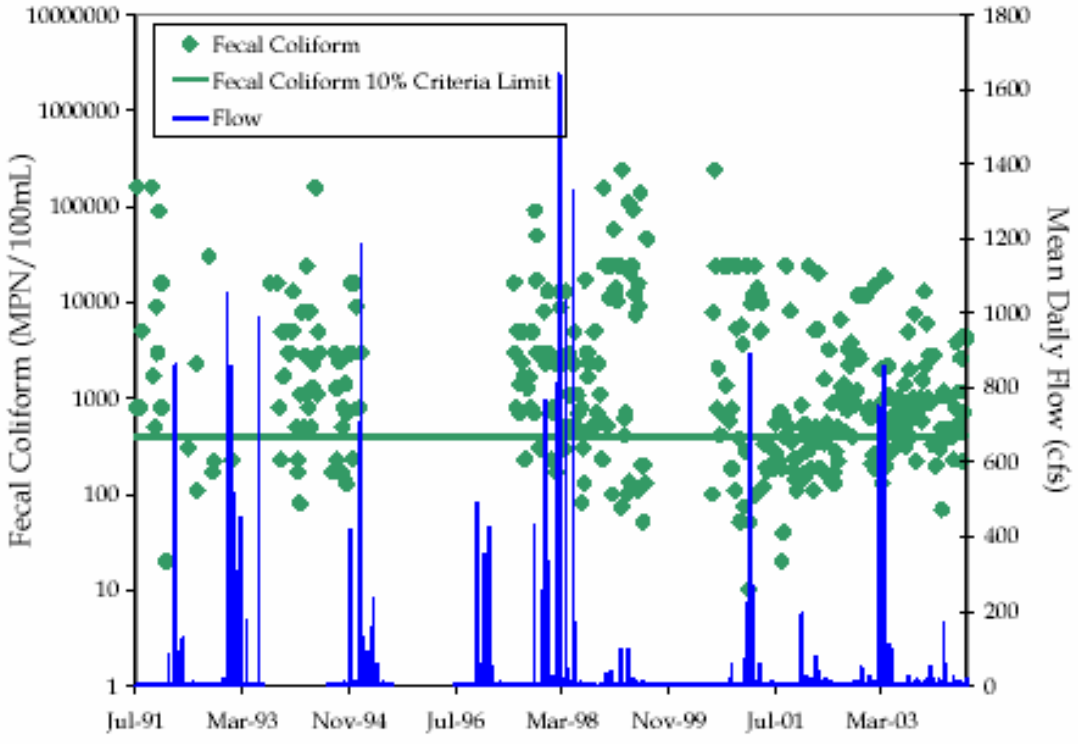


Figure 53
Time Series of Bacteria Concentrations and Flow in the Santa Ana Delhi Channel - Backbay

Figure B 4: Flow rate and bacteria concentration, Temescal Creek (CDM 2005 Figure 72)

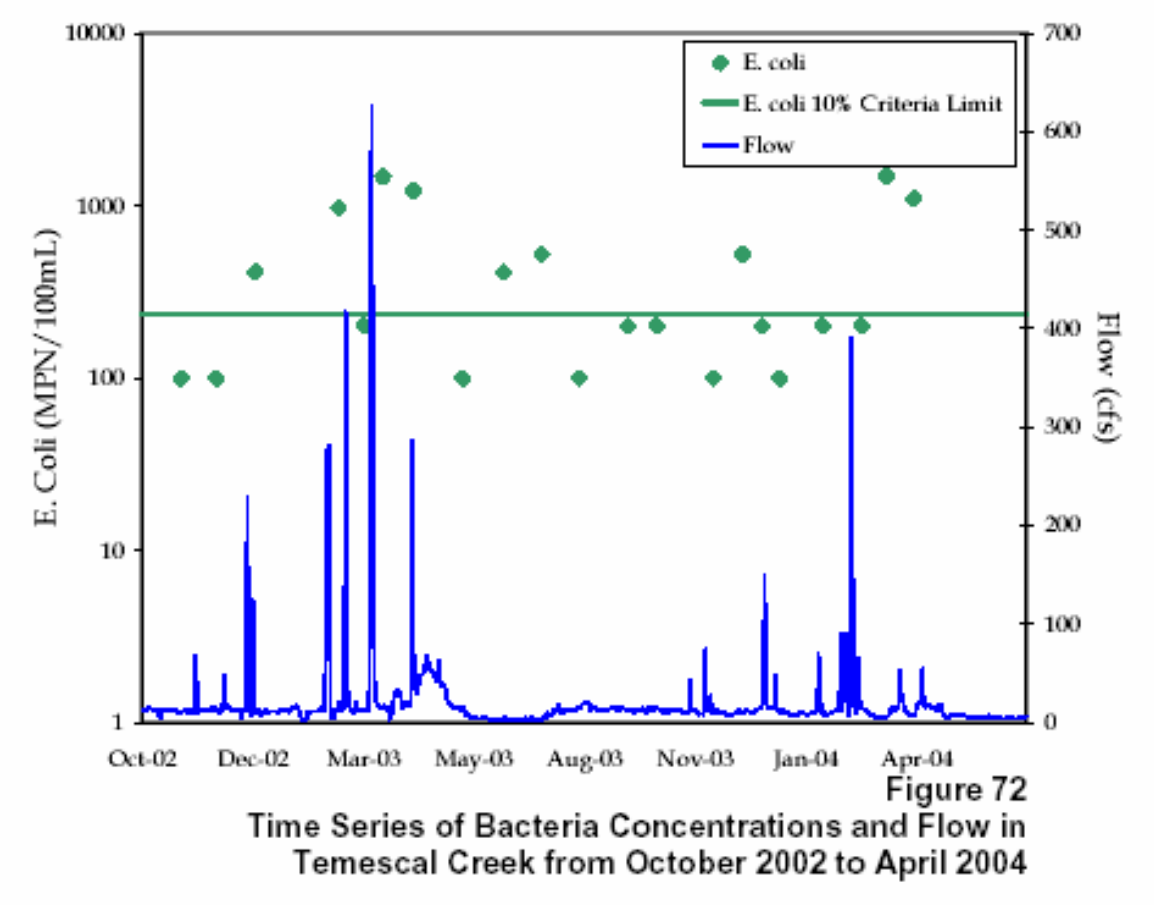


Figure B 5: Flow rate and bacteria concentration, Santa Ana River at MWD Crossing (CDM 2005 Figures 98 and 99)

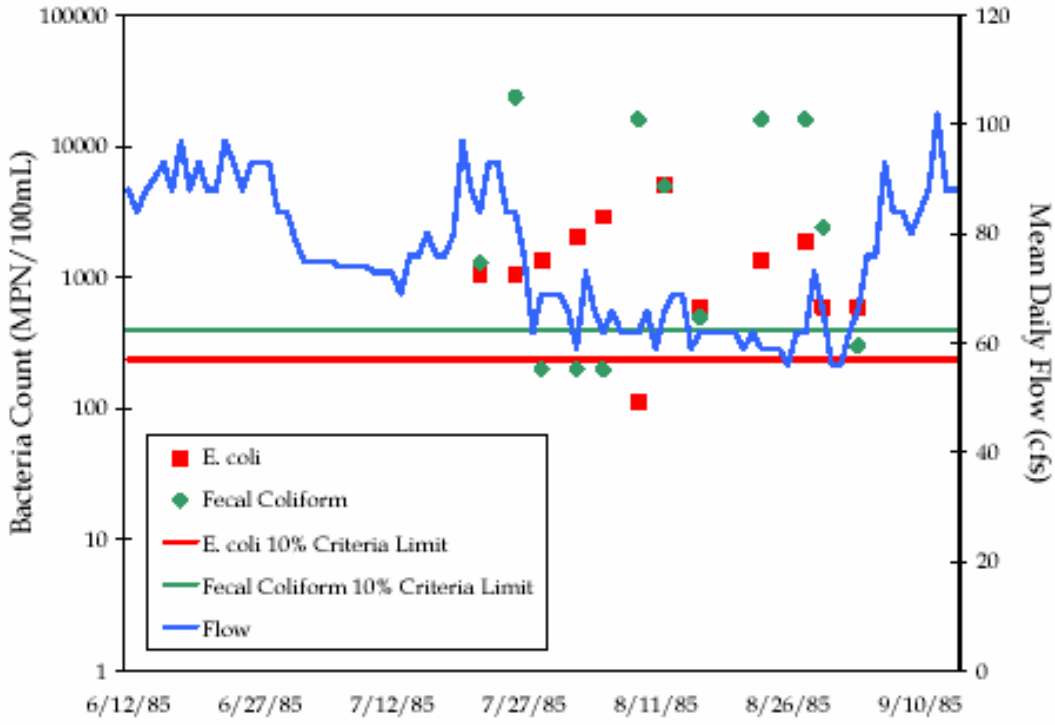


Figure 98
Time Series of Bacteria Concentrations and Flow in the Santa Ana River at the MWD Crossing Study Site

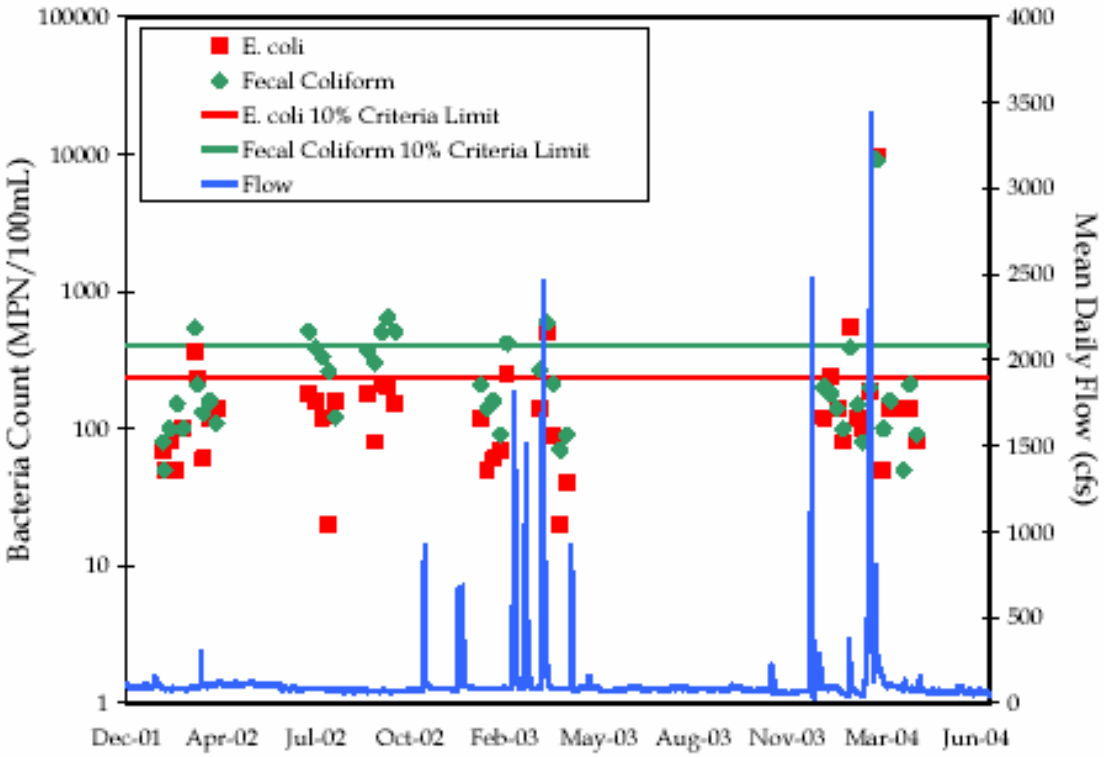


Figure 99
Time Series of Bacteria Concentrations and Flow in the Santa Ana River at the MWD Crossing Study Site

Figure B 6: Flow rate and bacteria concentration, Santa Ana River at Imperial Highway (CDM 2005 Figure 83)

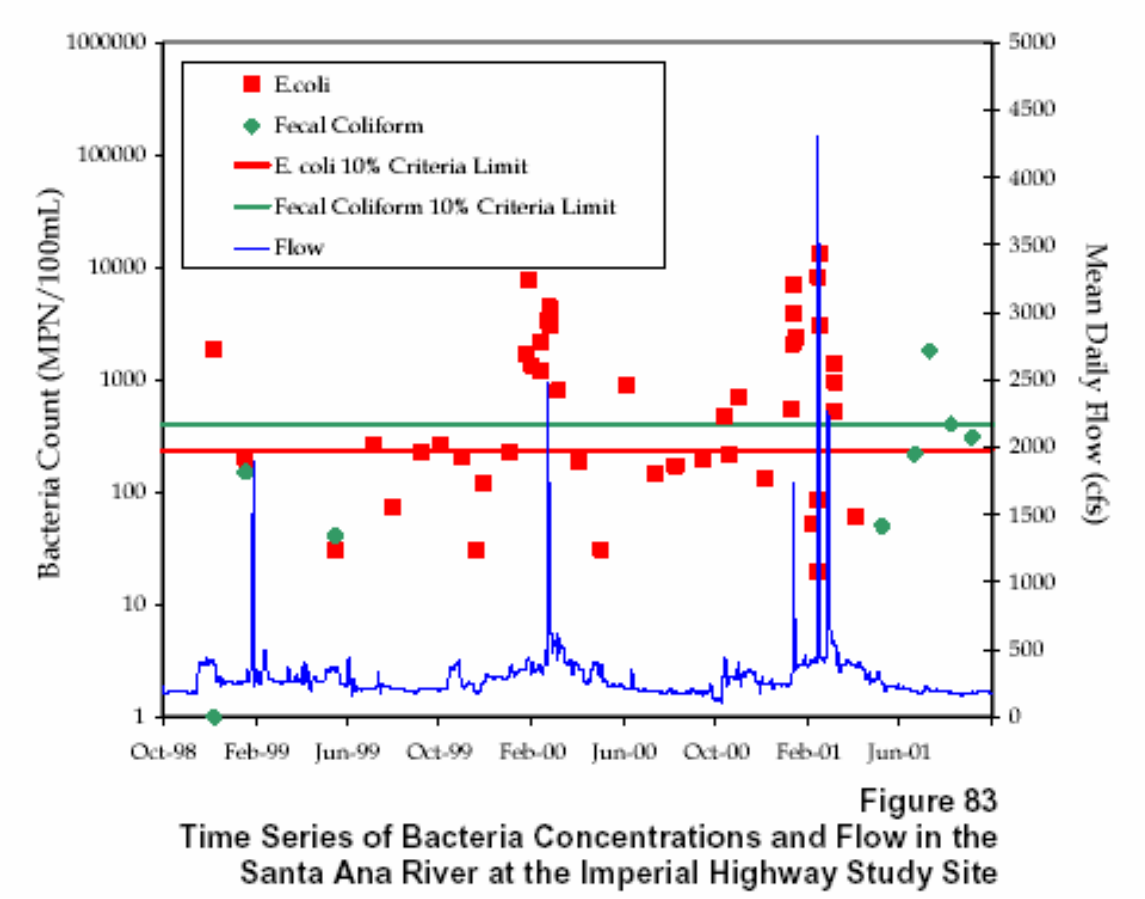


Figure 83
Time Series of Bacteria Concentrations and Flow in the Santa Ana River at the Imperial Highway Study Site

Figure B 7: Flow rate and bacteria concentration, Santa Ana River at Imperial Highway (CDM 2005 Figure s 84 and 85)

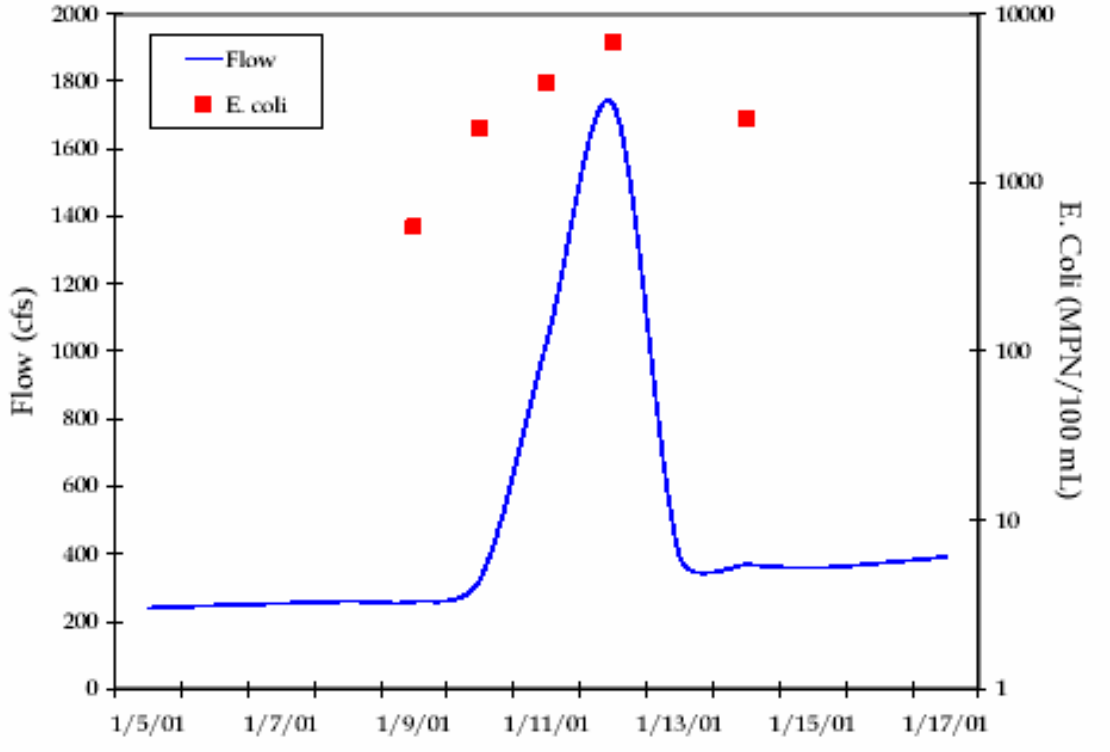


Figure 84
January 2001 Wet Weather E. coli Sampling Event

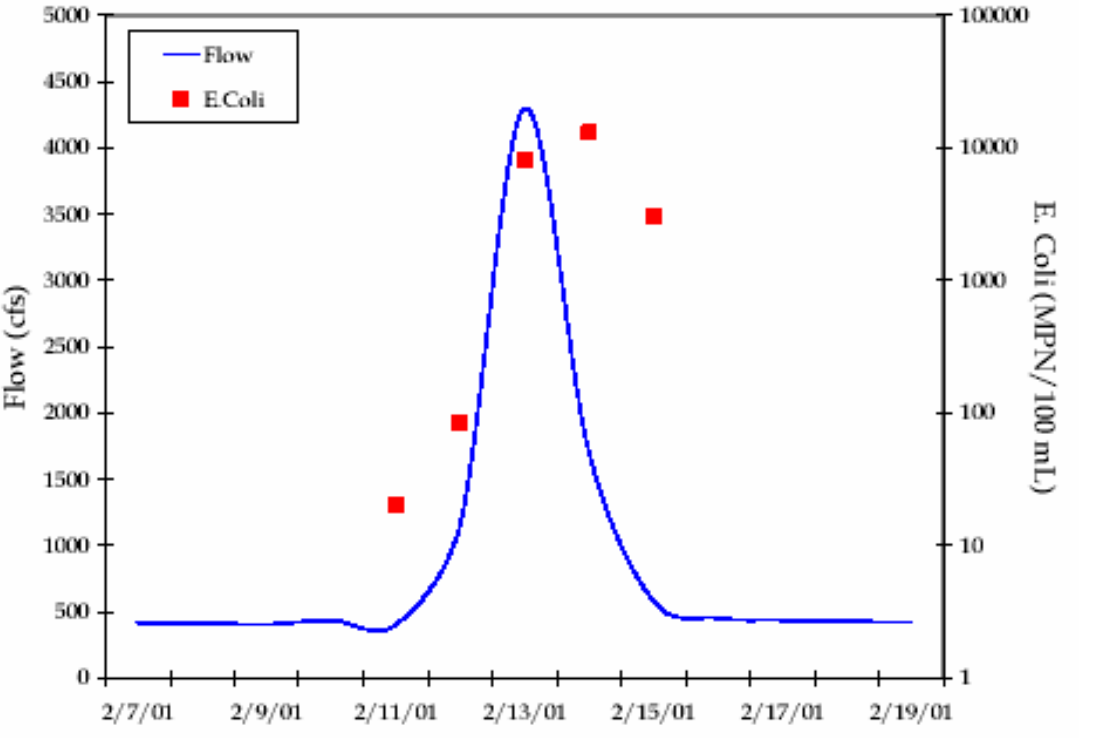


Figure 85
February 2001 Wet Weather E. coli Sampling Event

Figure B 8: Percent of months exceeding objectives (CDM 2005 Figure 102)

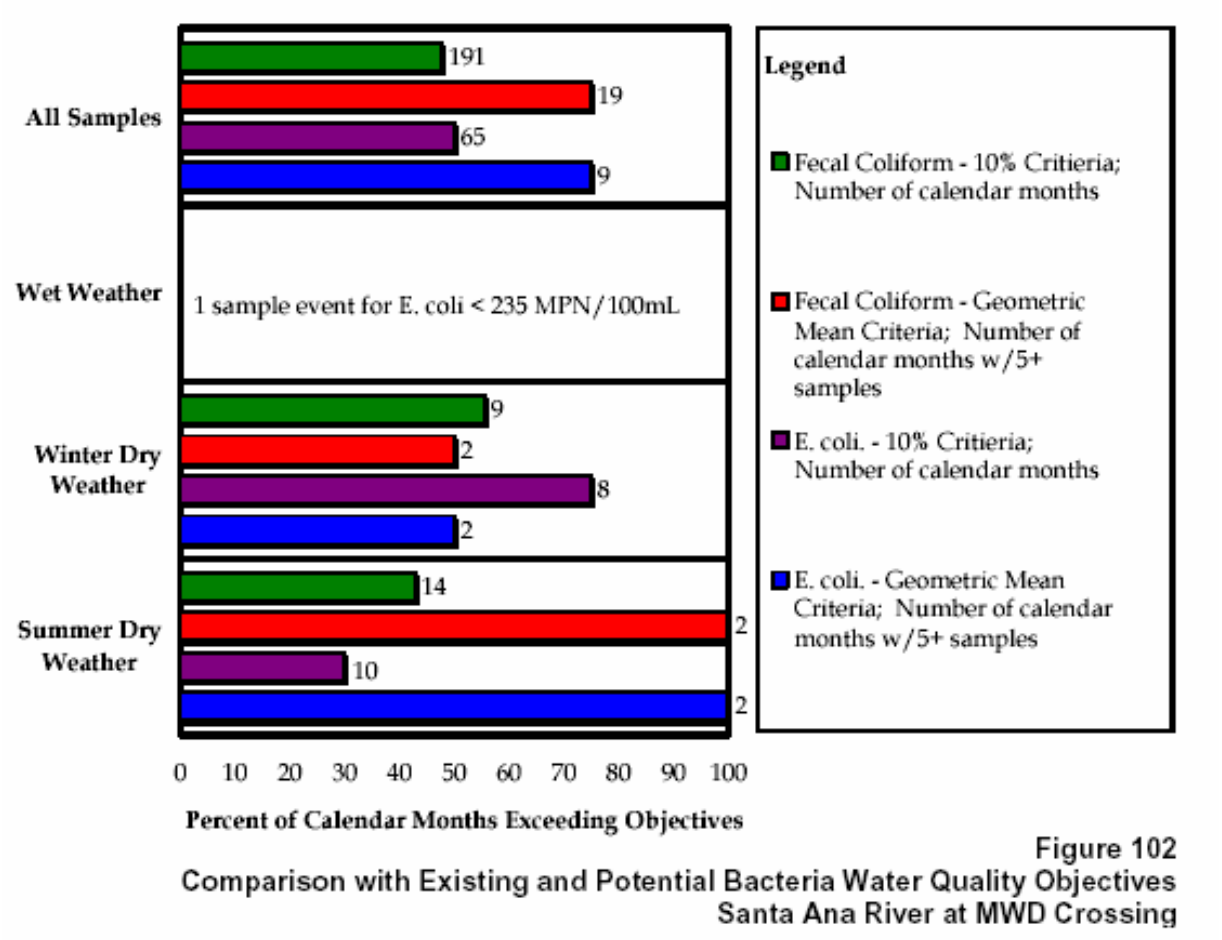


Figure 102
Comparison with Existing and Potential Bacteria Water Quality Objectives
Santa Ana River at MWD Crossing

Figure B 9: Percent of months exceeding objectives (CDM 2005 Figure 110)

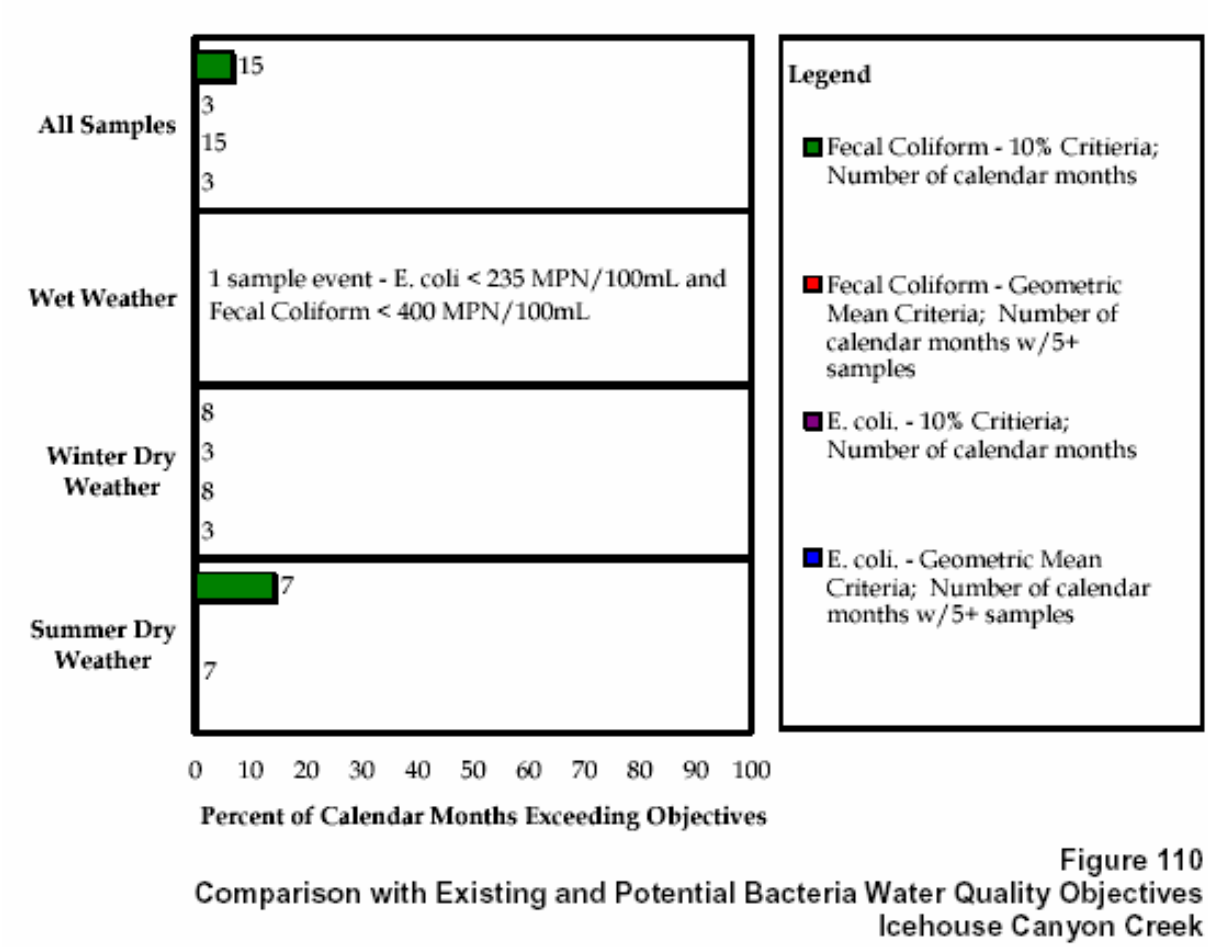


Figure 110
Comparison with Existing and Potential Bacteria Water Quality Objectives
Icehouse Canyon Creek

Figure B 10: Percent of months exceeding objectives (CDM 2005 Figure 88)

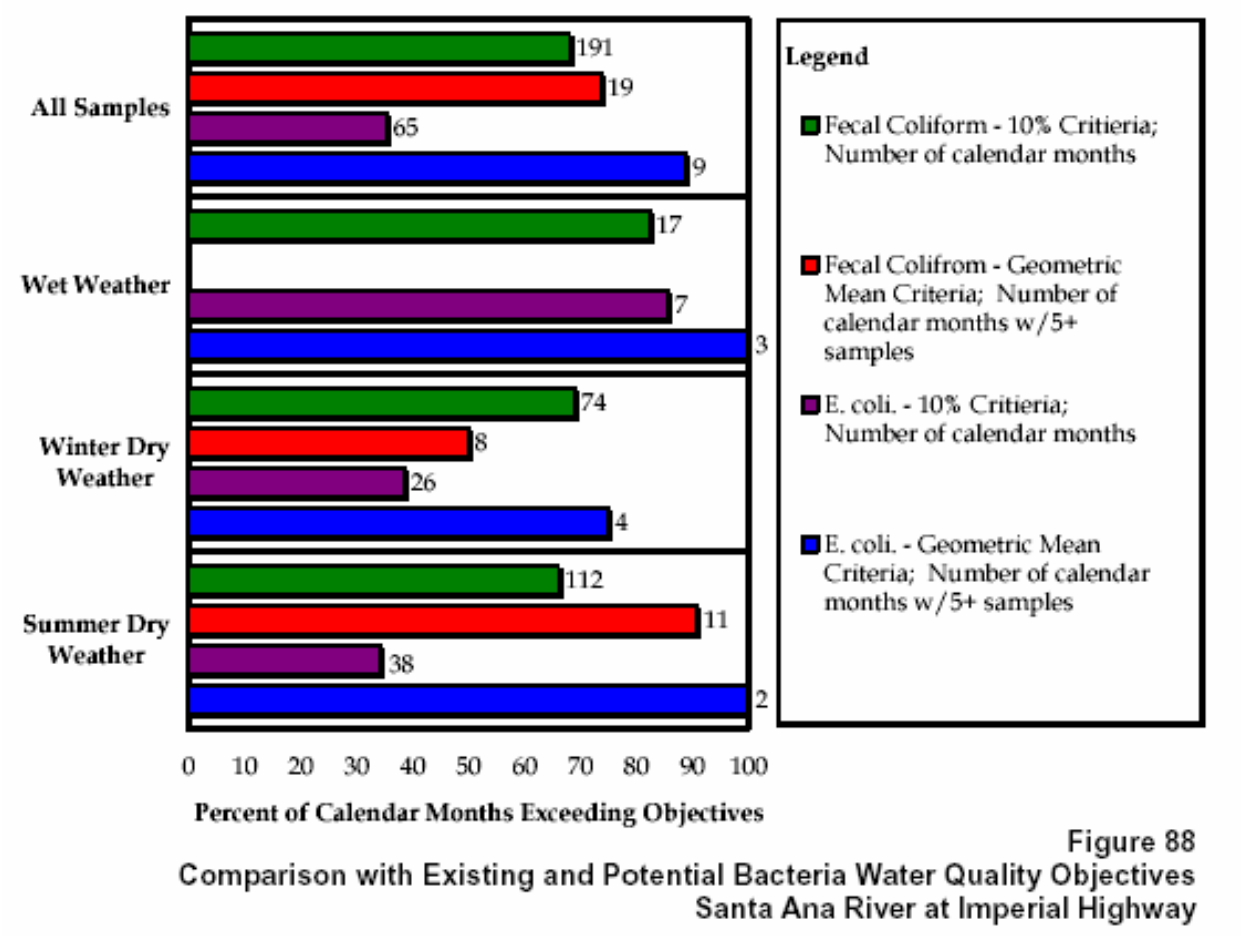


Figure 88
Comparison with Existing and Potential Bacteria Water Quality Objectives
Santa Ana River at Imperial Highway

Figure B 11: Percent of months exceeding objectives (CDM 2005 Figure 74)

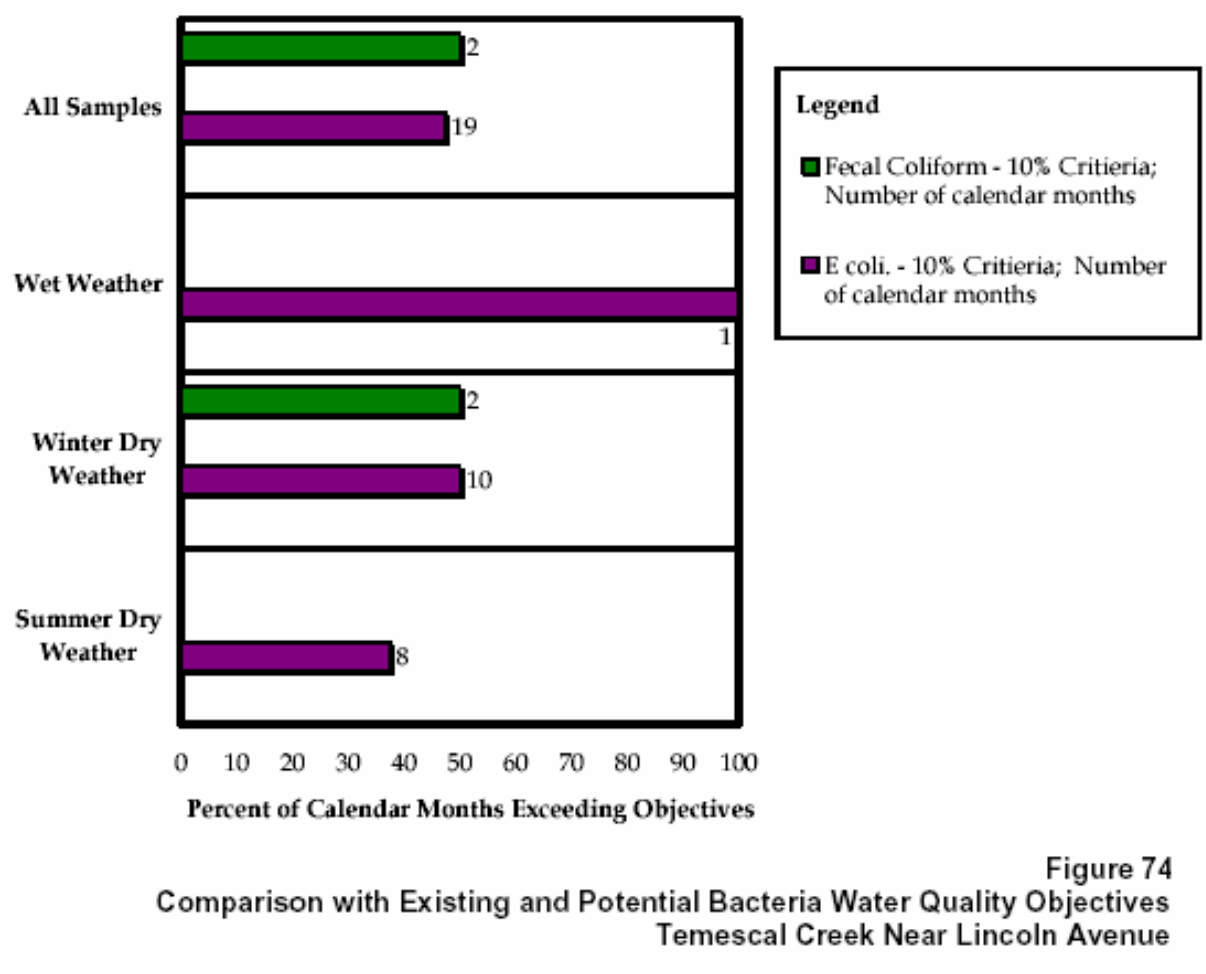


Figure 74
Comparison with Existing and Potential Bacteria Water Quality Objectives
Temescal Creek Near Lincoln Avenue

Figure B 12: Percent of months exceeding objectives (CDM 2005 Figure 38)

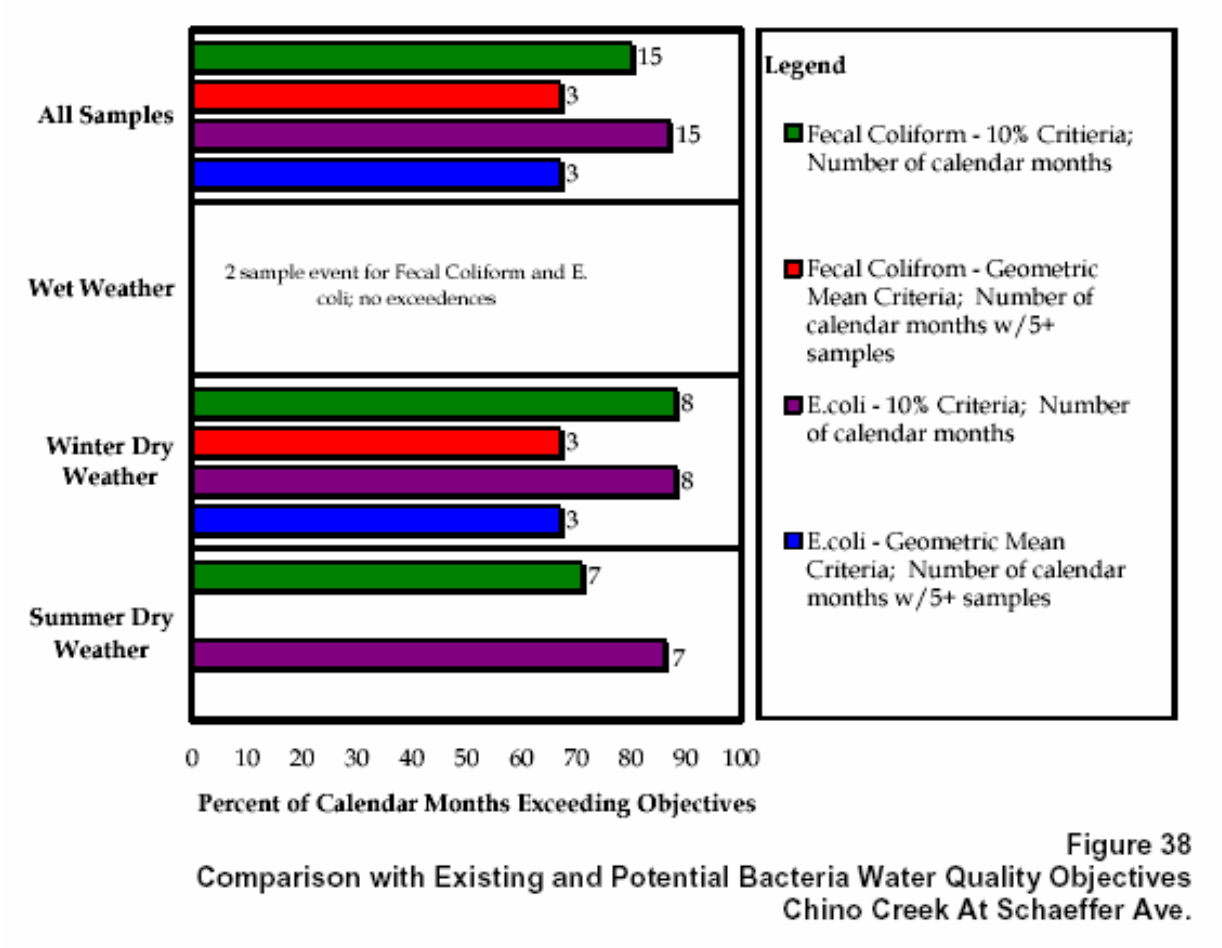


Figure 38
Comparison with Existing and Potential Bacteria Water Quality Objectives
Chino Creek At Schaeffer Ave.

Figure B 13: Percent of months exceeding objectives (CDM 2005 Figure 57)

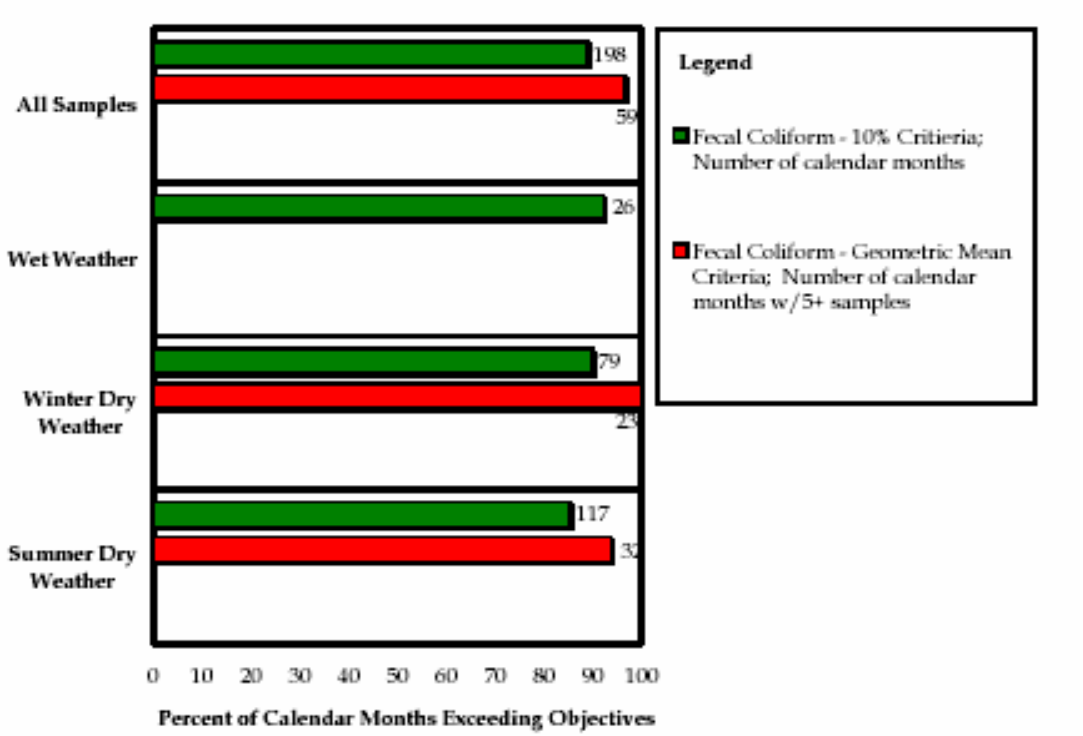


Figure 57
Comparison with Existing and Potential Bacteria Water Quality Objectives
Santa Ana Delhi Channel near Irvine Ave.

APPENDIX C

DATA FROM ALISO CREEK

G E O S Y N T E C C O N S U L T A N T S

838 SW First Avenue, Suite 530
Portland, Oregon 97204

(503) 222-9518
(503) 242-1416 Fax

MEMORANDUM

TO: SUSAN PAULSEN, FLOW SCIENCE

FROM: BRUCE WILLIAMSON, LISA AUSTIN, GEOSYNTEC CONSULTANTS

SUBJECT: ALISO CREEK BMP EFFECTIVENESS ANALYSIS

DATE: APRIL 13, 2005

CC: PETER MANGARELLA, GEOSYNTEC CONSULTANTS

Introduction

This purpose of this technical memorandum is to assess the efficacy of Best Management Practices (BMPs) installed in parts of Aliso Creek, Orange County, California (Figure 1) on the removal of pathogen indicators. Pathogen indicator data collected by Orange County Resources and Development Management Department in this watershed and on these BMPs has received increasing attention when project design features are evaluated by regulatory authorities. Therefore, it is important that we have a good understanding of these findings and their uncertainties.

The two BMPs assessed in this memo are:

1. Dry weather flows are passed through multimedia filtration/UV sterilization using a proprietary treatment unit 'Clear Creek Systems'. This treats low flow runoff from a two square mile catchment with mixed urban land use. The storm drain facility and catchment are designated as J01P28 in the watershed map and plans (Figure 1, 2B).

2. Wetland ponds to intercept watershed runoff and treat dry weather flow and first flush. These treat low flow and first flush runoff from a two square mile residential catchment. The storm drain facility and catchment are designated as J03P02 in the watershed map and plans (Figure 1, 2A).

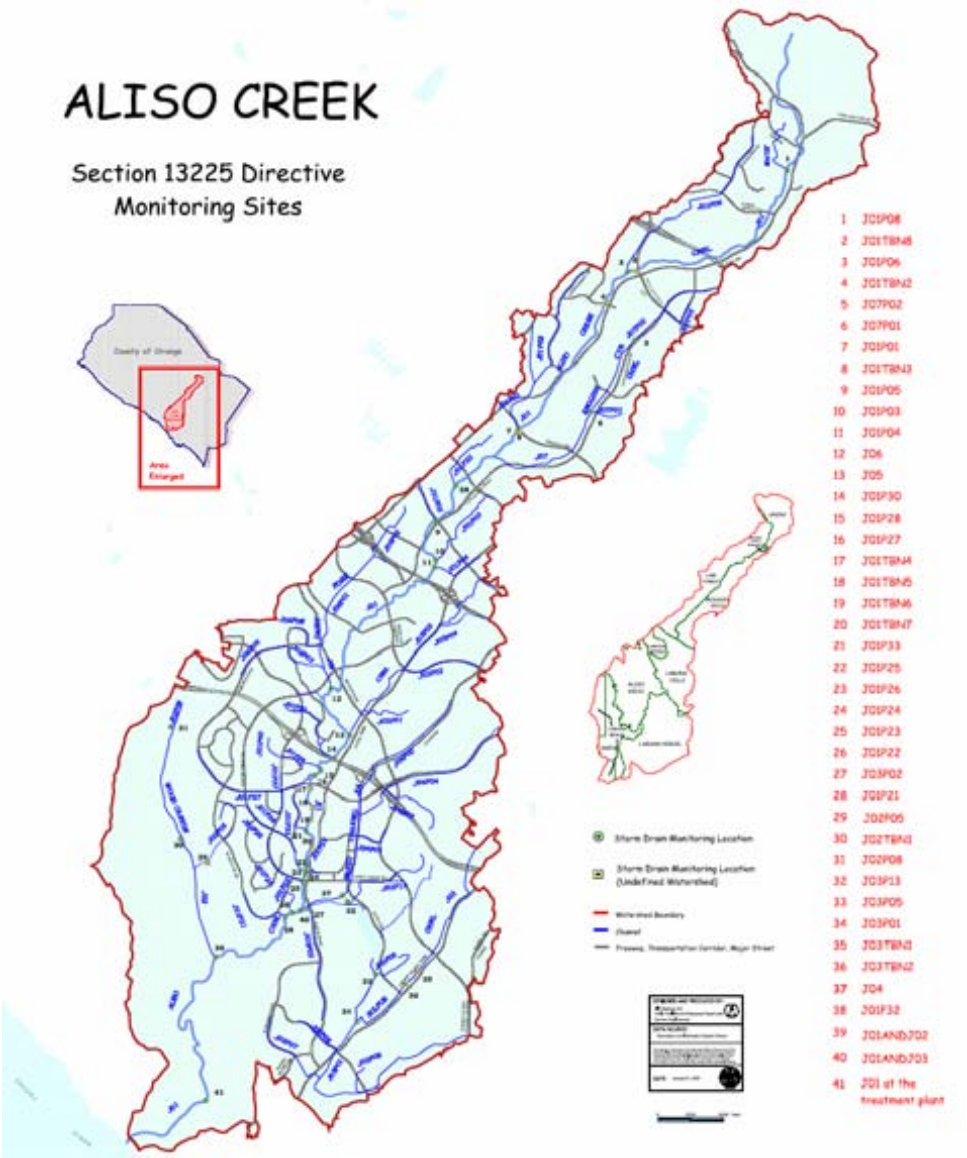
All monitoring of the BMPs and their receiving waters took place during dry weather. Consequently, low flows were mostly sampled, but during the wet season a proportion of these were probably elevated flows during storm recessions.

The data were collected by the County of Orange and its city partners and is available in reports listed at http://www.ocwatersheds.com/watersheds/Aliso_reports_studies.asp, and also in Evaluation Reports by the County of Orange.[1,2]

[1] County of Orange Resources and Development Management Department, Watershed and Coastal Resources Division. 'Aliso Creek Clean Beaches Initiative. Final Report for Agreement 01-227-550-0' submitted to Regional

Note that the Aliso Creek watershed Quarterly Progress Reports (QPR) refer to other BMPs installed in stormwater drains of urban watersheds at a number of locations in the Aliso Creek watershed. These include grassy swales for treating park runoff to Sulfur Creek in Laguna Niguel and a wetland biofilter in another branch of Sulfur Creek in Laguna Hills The status of the these BMPs is unclear, and no monitoring data for these BMPs were located in the QPR.

Figure 1



and State Boards in January 2005 and 'Wetland Capture and Treatment Final Report for Agreement No. 01-122-259-0' submitted to Regional and State Boards in March 2004.

[2] "Wetland Capture and Treatment Final Report for Agreement No. 01-122-259-0" submitted to Regional and State Boards in March 2004.

Figure 2A: Location of J03P02

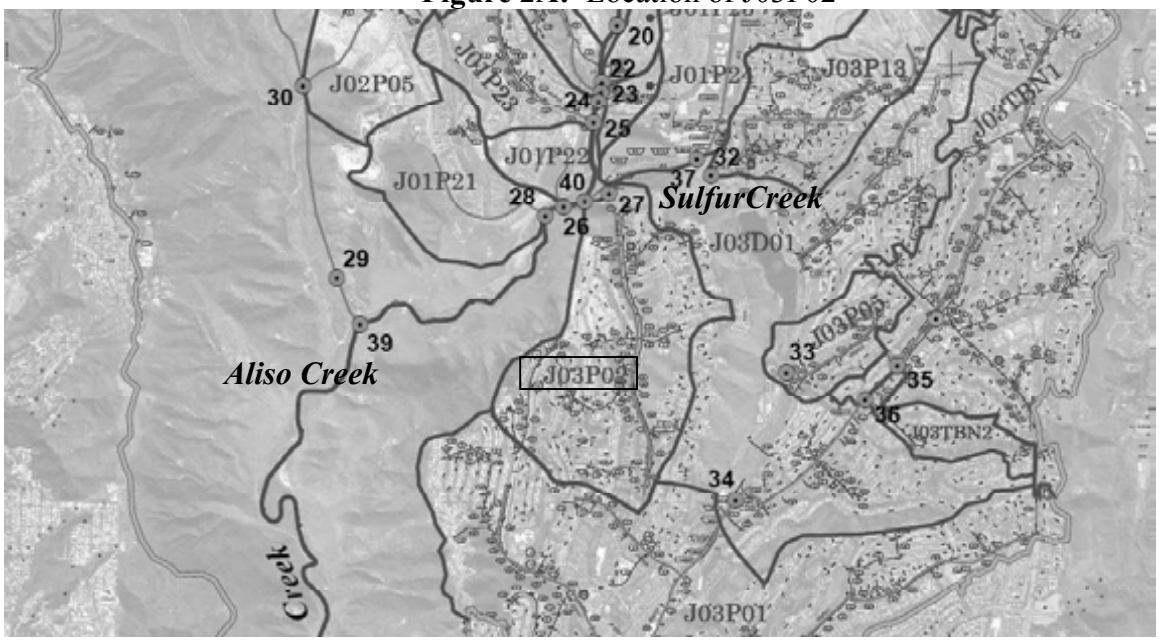
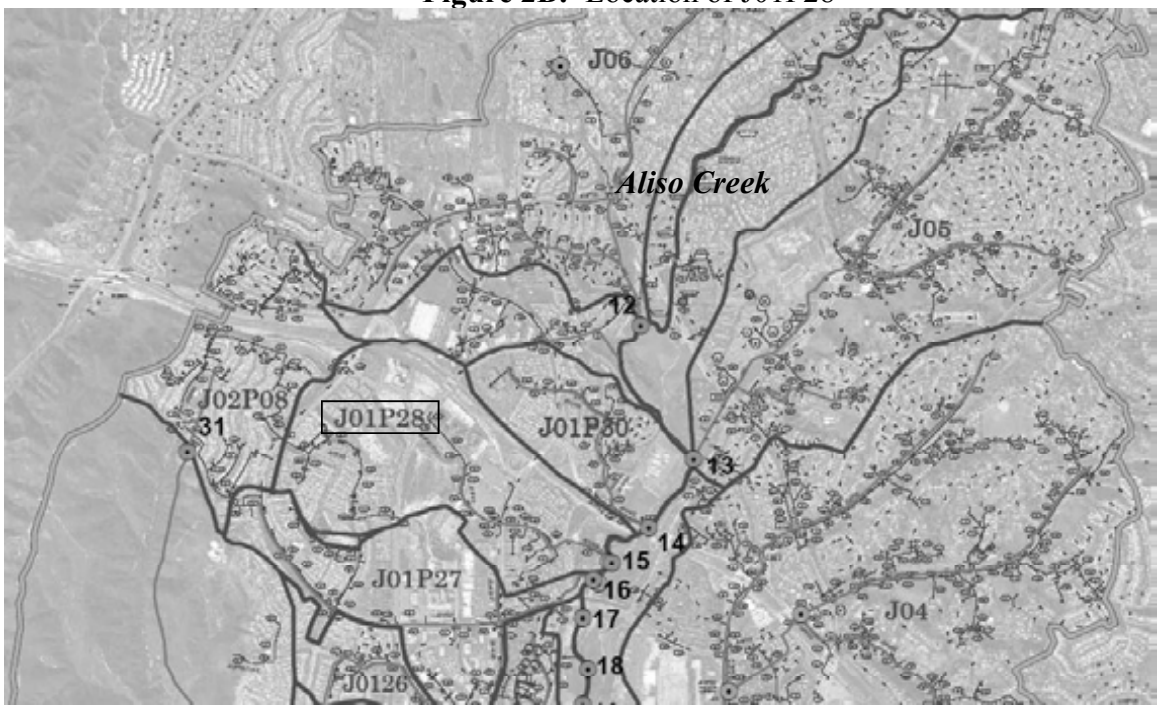


Figure 2B: Location of J01P28



Site Description

Aliso Creek Watershed

Aliso Creek watershed encompasses 30.4 square miles and includes portions of the cities of Aliso Viejo, Dana Point, Laguna Niguel, Laguna Woods, Laguna Beach, and Lake Forest. Its main tributary, Aliso Creek, originates in the Santa Ana Mountains inside the boundaries of the Cleveland National Forest. Smaller tributaries include Wood Canyon, Sulphur Creek, the Aliso Hills Channel, and English Channel (Figure 1).

Aliso Creek is the subject of a Directive issued by the San Diego Regional Water Quality Control Board (RWQCB) in 2001 for an investigation of urban runoff in the Aliso Creek watershed. The Directive found that the Permittees may be discharging waste with high bacteria levels from municipal storm drain outfalls into Aliso Creek and its tributaries. The Directive required the Permittees to begin a comprehensive monitoring program and undertake investigations within the storm drain system to identify the causes of the problem and the control actions needed to correct the problem. This has resulted in a comprehensive study involving weekly sampling of approximately 35 storm drains and their respective receiving waters, and numerous other initiatives in identifying sources and source control.

Part of the creek (J03P02) is subject to a Cleanup and Abatement Order (CAO) issued by the RWQCB in 1999. This was the result of a survey which showed that pathogen indicators (PI) in the drain were much higher than in Aliso Creek. Experience gained from the more comprehensive monitoring carried out since that time has shown that J03P02 is in the low to middle of the range of PI concentrations compared to the rest of the Aliso Creek watershed.

Sand Filtration/UV Sterilization

The J01P28 Interim Water Quality Improvement Package Plant BMP was executed in response to the San Diego RWQCB 13225 Directive to clean up Aliso Creek.

This treatment unit is located near the outlet of the J01P28 subcatchment (Figure 2). This subcatchment is a tributary to the main stem of Aliso Creek. The storm drain conveys runoff water from a fully developed area of approximately two square miles in the city of Aliso Viejo. Land uses in the catchment include residential, commercial, light industry, and parks. The BMP was installed in July 2003.

The CCS treatment system includes three multi media filters, two organo clay filters and two ultraviolet light disinfection chambers. The package plant treatment system has three main phases:

- Sediment and debris removal
- Oils, pesticides, and trace metals removal
- Disinfection

The larger debris and trash removal is performed at the inlet strainer that is located in an energy dissipation basin within the storm drain. Sediment removal is performed in the basin and in the multimedia filter. Oils, pesticides and trace metals are removed via adsorption onto the organo-clay media while the ultraviolet light chamber removes bacteria and viruses.

The package plant treatment system filters and disinfects approximately 100,000 gallons per day of urban dry weather runoff. The design capacity is 250,000 gallons per day. By October 2004, a total of 1.4 million gallons had been treated.

Monitoring results from the years 2001 through June 2003 were combined to form the "before" dataset, while results from August 2003 through December 2004 constituted the "after" dataset.

Once discharged from the unit, the water flows through a ponded area approximately 20 feet long, 6 feet wide and 1.5 feet deep, then 30 feet through a natural ditch to Aliso Creek. A monitoring site is located in the natural ditch 15 feet from Aliso Creek.

Wetlands

Wetlands have been installed near the outlet of subcatchment J03P28, which is a tributary to Sulfur Creek, itself a tributary to Aliso Creek (Figure 2A). The wetlands are positioned at the bottom of the catchment and designed to capture 100% of the low flows before they discharge to Aliso Creek. The catchment (538 acres) is entirely residential (1600 households, new to 30 years old). A number of structural BMPs have been implemented from 2000 to the present day.

1. From May 2000 to March 2001, dry weather flows were diverted to the AWMA Regional Sewage Treatment Plant.

2. From March 2001 to April 2003 (actually it is not clear when unit stopped operating), dry weather flows in the drain were treated by a mobile Clear Creek Systems filtration/UV treatment unit. The flow was diverted to the treatment plant (e.g., 15% of total flow in the July-September 2002 quarter) when the filter clogged or the UV malfunctioned.

3. The three wetlands were constructed progressively starting in about March 2001 and were completely online from April 2003[2].

J0302 has been subject to detailed studies because of the CAO. These include visual (video) inspection of sewer and storm drain pipes, field reconnaissance, resident surveys, flow monitoring, a wide range of upwatershed sampling and the identification the sources of the pathogenic indicator bacteria. Samples were examined for human enteroviruses, antibiotic resistance, and genotypes of E. coli. The researchers concluded that the primary sources of PI in J03P02 are not likely to be human, and are likely to be due to cows (soil fertilizer amendments), birds, rabbits, and some unidentified other animals. In the Aliso Creek QPRs, the Co-Permittees indicate that the following sources probably contribute to fecal coliform (FC) in J03P02:

- Organic soil amendments
- Turfgrass areas

- Wildlife
- Domestic pets
- Accumulated organic debris in the surface and subsurface storm drain system
- Street sweeping debris

The wetlands – called East, West and North, were positioned to capture 100% of catchment runoff during dry weather and first flush. Design features are summarized in Table 1. The hydrological network is outlined in Figure 3.

Wetland inflow is taken by intercepting flows in the stormwater pipes, including the 60-inch main pipe. After passing through the wetlands, some of the treated stormwater is routed back though the 60-inch pipe to an open channel just before its confluence with Sulfur Creek. Effluent from the West Wetland is discharged directly to this open channel, and does not pass through the pipe. Another untreated, unmonitored inflow also discharges to this point (Figure 2).

Table 1: Wetland design features (reference see footnote 2).

Wetland	Total Catchment Area (acres)	Planned intercepted area (acres)	Wetland Area (acres)	Depth (ft)
East	374	37	0.3	1
West	342	312	0.69	0.5
North	122	122	0.3	1

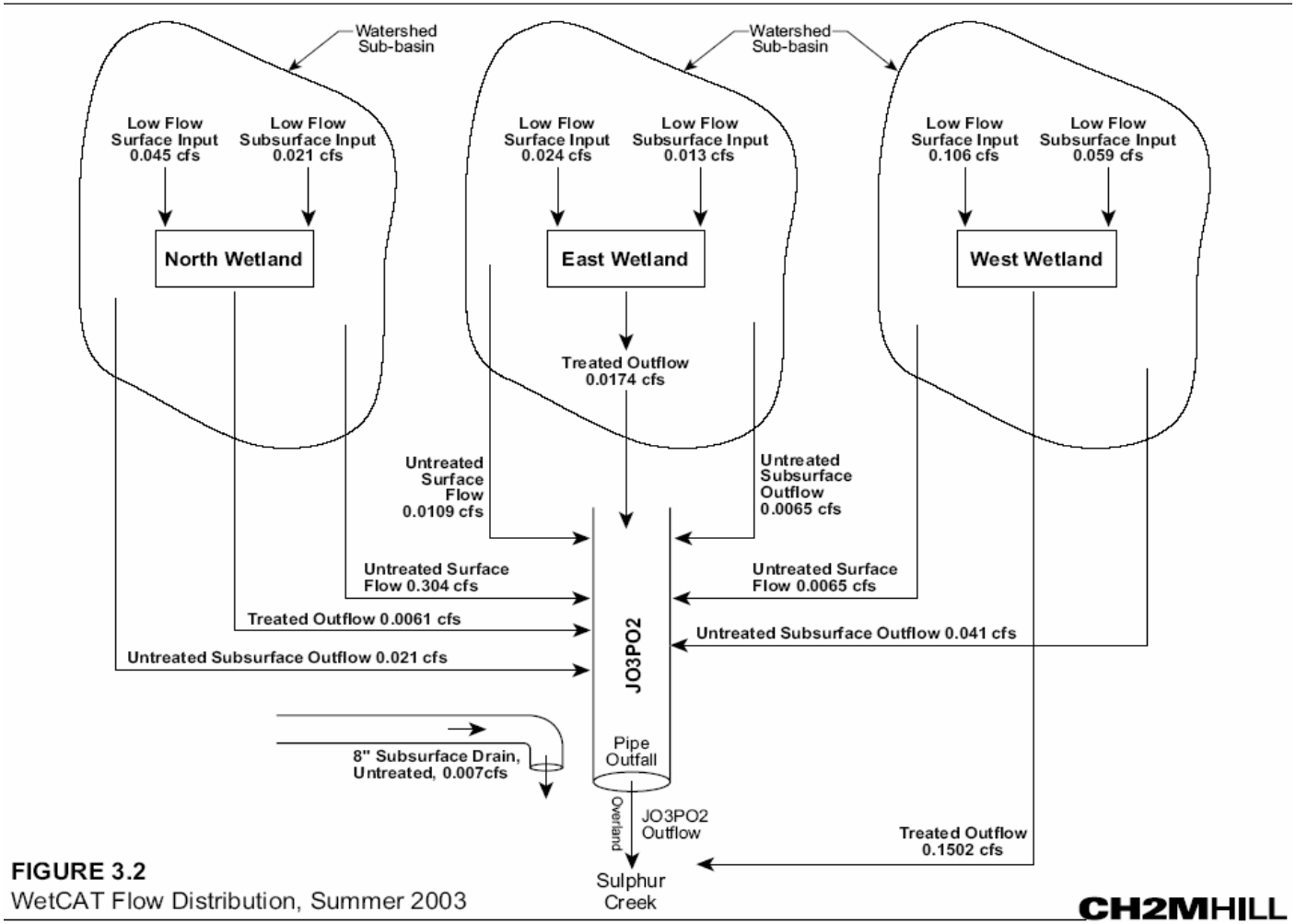
Sampling Procedures

All sampling was conducted during "dry weather," which is defined as no rain on the day of sampling. Sampling was conducted under strict protocols (see Aliso Creek 8th Quarterly Progress Report). Quality Assurance/Quality Control (QA/QC) sampling procedures were implemented that should have prevented contamination during sampling and significant changes to the sample during transport to the laboratory.

Directive Monitoring: Each location has three monitoring sites: two of these are on the main stem, 25 feet upstream and downstream of the storm drain discharge, the other is on the storm drain itself, approximately 15 feet above its confluence with the stream. These three sites were monitored weekly, so that at least five samples were collected each month, at random intervals. Some of these monitoring sites are shown in Figure 1.

BMP Monitoring: In addition to the directive sampling program, the influent and effluent to the BMPs were monitored.

Figure 2. Source: Wetland Capture and Treatment Final Report (2004)[2]. Note: untreated Surface Flow from North Wetland should probably be 0.0304 cfs.



Summary of Monitoring Results

J01P28 - Multimedia Filtration/UV Digestion

Influent/effluent. Comparison of the influent and effluent concentrations demonstrates a 99.6% reduction in fecal coliform levels. The geometric mean decreases from 77,414 CFU/100mL to 317 CFU/100mL.

Stream and drain monitoring. A statistical analysis of the levels in the receiving water (the "directive" dataset) is summarized in Table 2 and as box plots in Figure 3-4. These refer to all data collected before BMP installation. The County monitoring reports summarize data for quarterly monitoring periods. In the QPR, quarterly monitoring data are compared between years to reduce variance from seasonality, and constitute a more powerful assessment of the data. However, for our purposes here, the lumped data is sufficient to demonstrate their findings.

Table 2: Comparison of geometric means (cfu/100 ml) before and after multimedia filtration/UV sterilization. The BMP is installed about 35 feet upstream of the storm drain monitoring site.

Locations	**TC**		**FC**		**ENT**	
	before	after	before	after	before	after
u/s	5353	2851	775	773	990	662
storm drain	52267	15232*	14633	5827*	9171	1401*
d/s	17248	5142*	2722	1696*	1791	839*

* = significant change (1-way ANOVA, $\alpha<0.05$)

Regrowth. Comparison of effluent and the 'directive' storm drain monitoring site, show a large increase in FC levels in the approximately 35 feet between the unit discharge and the storm drain monitoring site. No other discharges were found, which suggest that rapid re-growth has taken place in the water column, or re-infection has occurred from sloughing or resuspension of bacteria from immersed channel-side vegetation, organic debris and/or sediments. The geometric mean increases in this short distance from 317 cfu/100mL to 2,575 cfu/100mL.

Further work is planned by the County on the re-growth issue. Permits have been requested to perform clean up work on the habitat and the storm drain outlet basin.

Figure 3: FC levels for J01P28 monitoring site.

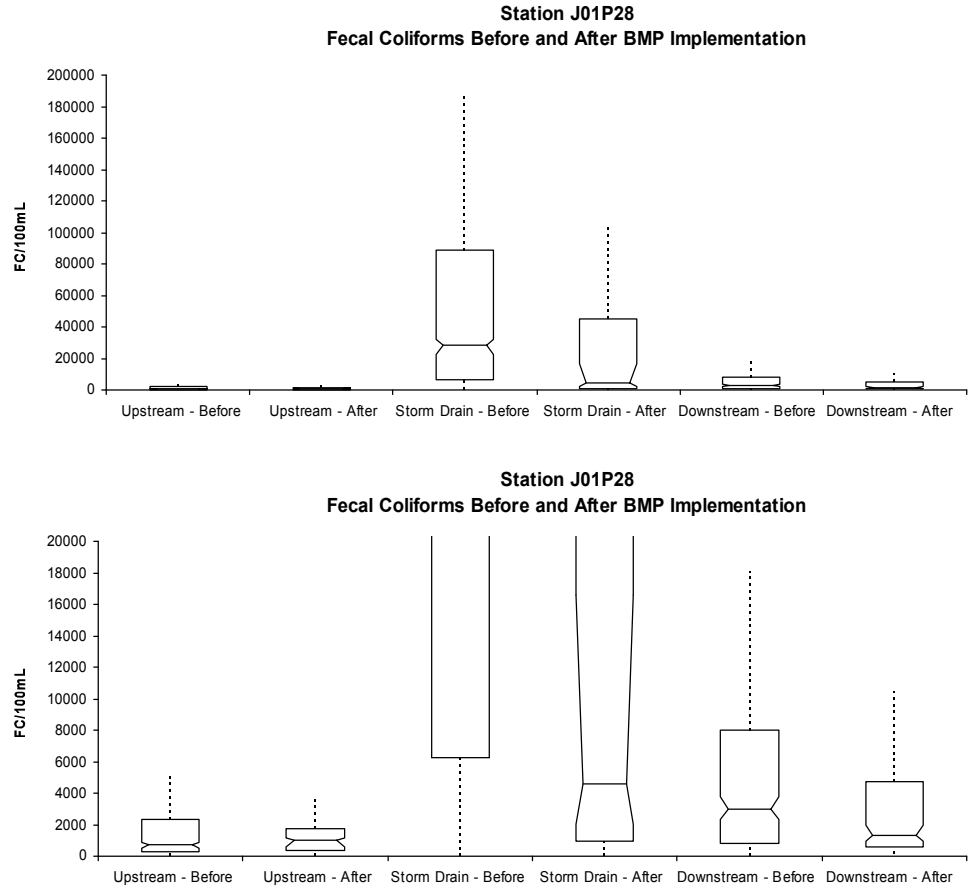


Figure 4: ENT levels for J01P28 monitoring site.

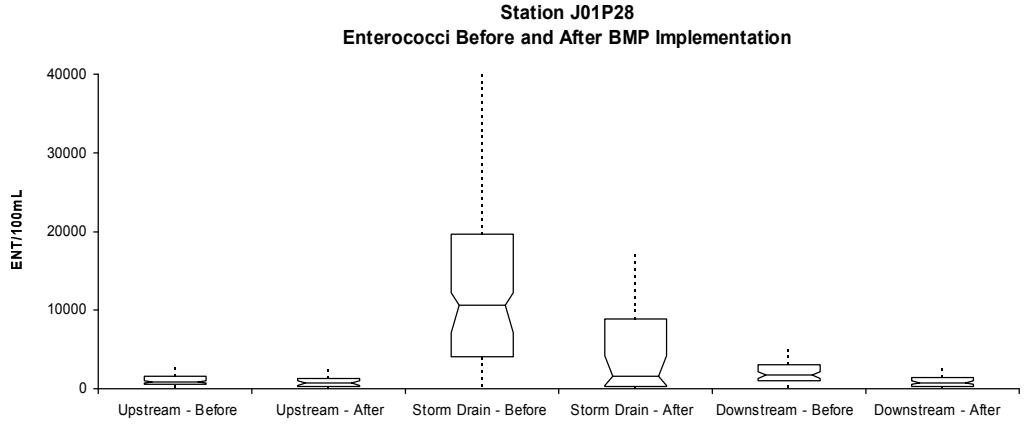
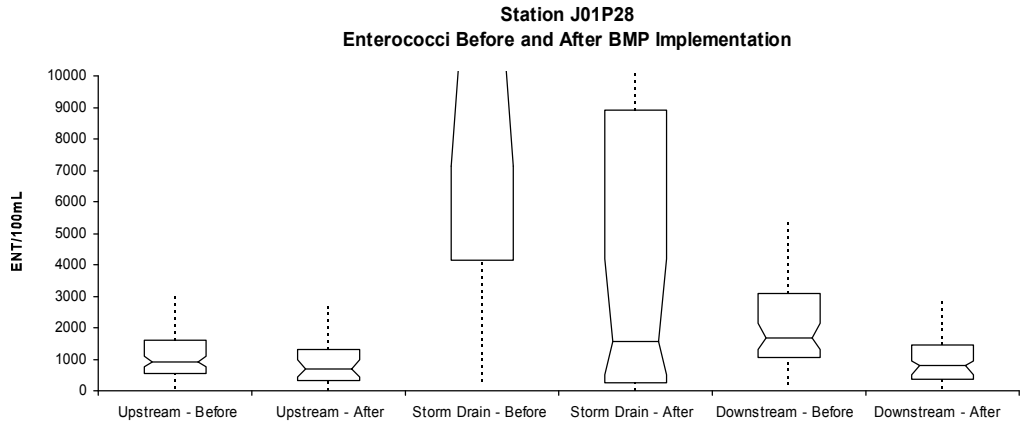


Figure 4 (continued)



J03P02 – Wetland BMPs

Influent/effluent. All monitoring took place during dry weather. Flows were measured, but only once per month and not for each sampling occasion. Most sampling took place at lows flows. The flow was typically 0.25 cfs with a range of 0.13-0.56 cfs.

Wetland monitoring in the three wetlands showed 90 to 99 percent reduction in FC levels from 2001 to present day (e.g. see Table 3). (Note that the three wetlands were installed and monitored progressively – results from 2001 were from one wetland only). Overall, 90 percent of treated effluent samples met the REC-1 objectives for FC. Although enterococci (ENT) levels dropped by 60 to 99 percent in wetlands, wetland effluent did not meet the steady state objective of 33 cfu/100ml during the period of monitoring (2001-2004). Few individual wetland samples met the single-sample objective.

Table 3: East Wetland fecal coliform (cfu/100mL) removal March 2001 – August 2002.

Parameter	Inflow	Outflow	Removal
Median	5000	50	99%
Mean	14900	150	99%
Geometric mean	2,800	35	99%

Overall there has been a progressive decline in FC and ENT since the wetlands have progressively come on line.

As well as the wetland monitoring, the effluent from the mobile UV sterilization unit was monitored when it was installed (between March 2001 to April 2003). The influent was not monitored directly. A cursory scan of the results suggests that the treatment unit effluent quality met REC-1 requirements on most months, but failed at times, which was attributed to the sand filter clogging.

Stream and drain monitoring. No "before BMP implementation" could be found because the 'directive' monitoring period encompassed either diversion to the sewage treatment plant, UV sterilization and/or wetland treatment. (However, some data is available somewhere, because it led to the CAO).

The dry weather discharge from the storm drain had little or no effect on the FC levels in Sulfur Creek. The flow from J03P02 is about 10 percent of the flow in Sulfur Creek.

The bacterial quality of the J03P02 storm drain discharge has steadily improved over the monitoring period. However, the improvement is quite complex, as described in the following section.

Re-growth. There is evidence that re-growth occurs between the wetlands and the storm drain monitoring sites. The concentrations in the open channel at the end of the pipe are about twice what is expected based on mass flow considerations.

However, there are some ambiguities in the various Quarterly Reports about the nature of the connection between the catchments, wetlands, and the J03P02 monitoring site[3]. This has been resolved in the detailed report on the BMP project for J03P02[2]. Measurements show that a high proportion of the flow is not intercepted (about 37 percent). Figure 2 also shows that the largest wetland ('West') bypasses and discharges downstream from the pipe.

Therefore, the apparent re-growth phenomenon could be wholly or partly due to the "recontamination" by the un-intercepted flows from the catchment. The project investigated this by carrying out a mass balance calculation. Unfortunately the report does not give any details on the calculations, but states that concentrations at the end of the pipe after discharge are about twice what is expected based on these mass flow considerations.

GeoSyntec confirmed that there was about this order of magnitude difference between observed and calculated mass flows using flows given in Figure 2 and using appropriate median FC numbers for the summer 2003 monitoring period. However, the proposition of re-growth, while plausible, is uncertain because:

- There is a significant input of untreated surface and subsurface flows into and at the end of the J03P02 pipe
- Most flows were estimated and not measured
- Many of the FC and ENT concentrations used in the mass flow calculations were not measured and assumed values were taken from the monthly monitoring data.
- There is a high degree of variability in monitored FC and ENT

The rates of this apparent re-growth appear to be seasonal and variable. As described above, usually observed levels at the J03P02 monitoring site are higher than the combined flows from the wetland. Fecal coliform and enterococci increase by about 100 percent in-pipe during spring, summer, and fall. However, this apparent re-growth does not occur during winter months and

[3] Most comments imply a 200 foot pipe, but 14th QPR refer to pipe outlet and 200 feet overland distance.

sometimes die-off can be observed. For example, the winter FC levels in 2004 were 1/8th of those predicted from the combined treated and untreated contributions, while ENT levels are about the same as predicted levels. The report suggests that die-off and re-growth (or re-contamination) of ENT and FC may be temperature and salinity dependent.

The overall findings of the BMP study to this particular watershed is that as the BMPs came on line, there was a steady improvement in the quality of the J03P02 discharge to Sulfur Creek during some seasons[4]. Results from monitoring the drain downstream of the BMPs show:

- Spring (Apr-Jun) geomeans for FC fell from 2001-2003. The 2004 geomean was similar to that for 2003.
- Summer (Jul-Sep) geomeans for FC have not fallen with statistical significance
- Winter (Jan-Mar) geomeans for FC fell from 2002 – 2004.

Discussion and Conclusions

Filtration coupled with UV sterilization reduced indicator bacteria to below the REC-1 standard. This was demonstrated at both sites. However, the benefits are compromised by what appears to be re-growth. At J01P28, the re-growth/re-inoculation occurred in a natural steam reach consisting of a pool and run, which was shaded with riparian vegetation dangling in the stream. It occurred within only 35 feet of the discharge point from the treatment unit.

Wetlands reduced fecal coliform (FC) levels by 90 to 99 percent to below the REC-1 guideline for 90 percent of the samples. They also reduced enterococci (ENT) levels by 60 to 99 percent, but the effluent from the three wetlands always exceeded the steady-state ENT objective, and usually exceeded the single sample objective. As with J01P28, the benefits of wetland treatment were compromised by the low-flow capture rate and what appears to be re-growth or re-contamination after discharge from the BMPs. Concentrations of FC and ENT increase between the wetland effluent and the J03P02 monitoring site 15 feet from its confluence with Sulfur Creek. The summary report proposed that most of the re-growth/re-inoculation occurred within a 200-foot pipe carrying wetland effluent to the confluence with Sulfur Creek.[2]

The study report proposed that re-growth was plausible because there was opportunity and time for re-growth to occur. The combined effluent from the East and North wetland is conveyed to Sulfur Creek through the pipe, which has a transit time during low flow of 15 minutes. As stated in the Wetland Capture and Treatment Final Report 2004[2] "Given ….. the microbiologists 'rule of thumb' that bacterial populations can double every 15 minutes under ideal conditions, rapid in-pipe propagation of FC and ENT in the dark pipe may be the main factor, or may be combined with recontamination from bioslimes or muck deposits" (Clean-Up & Abatement Order 99-211 17th QPR). Another possible reason is that the structures which divert low flow from the stormwater pipes to the wetland also trap and retain organic debris, which may act as substrates

[4] This is somewhat surprising given that the drain water was treated by multimedia filtration/UV disinfection or diverted to the sewer system while the wetlands were constructed.

for re-growth. However, re-contamination by unmonitored inflows may also be partly or wholly responsible for the observed increase between the BMPs and the confluence.

The results suggest that the benefits of BMPs may be compromised by re-growth, which occurred in both the natural channel and pipe downstream of the monitored BMPs. The various investigators have concluded that treatment systems would need to be positioned at the bottom of the watershed directly before discharge to the receiving water body – mainly to prevent regrowth during warm weather conditions.[1] Another important general conclusion in the study (see City of Laguna 6th QPR Aliso Creek 13225 Directive) states 'that "primary" bacteria concentrations (from direct deposits of bird droppings, for example) in runoff can be magnified by the "secondary" propagation of bacteria populations within the environment, so that *controlling propagation* may ultimately become as important as *source reduction* in reducing overall outfall concentrations. The research results also suggest that the presumption of a statistically valid relationship between certain concentrations of fecal coliform and an acceptable vs. unacceptable magnitude of public health risk (which is the basis for the REC-1 and REC-2 objectives) may be seriously flawed.'

The proposition that re-growth occurs after treatment has wide ranging implications for stormwater management. Given the uncertainties outlined above as to whether re-growth occurs after wetland treatment, the County study results should be confirmed by more detailed studies and sampling, such as:

- more frequent sampling of concentrations taking into account time of travel
- stormwater runoff monitoring (not just dry weather flows)
- measurement of flows where possible.

It is unknown whether the re-growth phenomenon apparent at the Aliso Creek sites would result in much higher concentrations over longer distances, but such an experiment cannot be conducted at the County-selected sites.

Finally, it is re-emphasized that monitoring was only conducted during dry weather conditions – mostly low flow and do not reflect storm runoff conditions, except for possibly occasionally during the storm regression phase. The impact of storm runoff on the treatment efficacy of the BMPs tested at Aliso Creek is unknown. Likewise, it is unknown what impact high flow may be on the mechanisms that lead to re-growth or re-inoculation; such flows may deliver organic debris and sediments and also slough off slimes and accumulations of organic detritus.

APPENDIX E

EFFECT OF URBANIZATION ON GROUNDWATER RECHARGE IN THE SANTA CLARITA VALLEY

Effect of Urbanization on Aquifer Recharge in the Santa Clarita Valley

TO: Tom Worthington/Impact Sciences, Inc.

FROM: John Porcello/CH2M HILL

DATE: February 22, 2004

Introduction

In a groundwater basin, the effect of urbanization on recharge to underlying groundwater is dependent on land uses, water uses, vegetative cover, and geologic conditions. Groundwater recharge from undeveloped lands occurs from precipitation alone, whereas areas that are developed for agricultural or urban land uses receive both precipitation and irrigation of vegetative cover. In an urban area, groundwater recharge occurs directly beneath irrigated lands and in drainages whose bottoms are not paved or cemented. This memorandum discusses the general effects of urbanization on groundwater recharge and the specific effects in the Santa Clarita Valley.

Summary of Findings

In the Santa Clarita Valley, stormwater runoff finds its way to the Santa Clara River and its tributaries, whose channels are predominantly natural and consist of vegetation and coarse-grained sediments (rather than concrete). The stormwater that flows across paved lands in the Santa Clarita Valley is routed to stormwater detention basins and to the river channels, where the porous nature of the sands and gravels forming the streambeds allow for significant infiltration to occur to the underlying groundwater.

Increased urbanization in the Valley has resulted in the irrigation of previously undeveloped lands. The effect of irrigation is to maintain higher soil moisture levels during the summer than would exist if no irrigation were occurring. Consequently, a greater percentage of the fall/winter precipitation recharges groundwater beneath irrigated land parcels than beneath undeveloped land parcels. In addition, urbanization in the Santa Clarita Valley has occurred in part because of the importation of State Water Project (SWP) water, which began in 1980. SWP water use has increased steadily, reaching nearly 44,500 acre-feet (AF) in 2003. Two-thirds of this water is used outdoors, and a portion of this water eventually infiltrates to groundwater. The other one-third is used indoors and is subsequently routed to local water reclamation plants (WRPs) and then to the Santa Clara River (after treatment). A portion of this water flows downstream out of the basin, and a portion infiltrates to groundwater.

Records show that groundwater levels and the amount of groundwater in storage were similar in both the late 1990s and the early 1980s, despite a significant increase in the

urbanized area during these two decades. This long-term stability of groundwater levels is attributed in part to the significant volume of natural recharge that occurs in the streambeds, which do not contain paved, urban land areas. On a long-term historical basis, groundwater pumping volumes have not increased due to urbanization, compared with pumping volumes during the 1950s and 1960s when water was used primarily for agriculture. Also, the importation of SWP water is another process that contributes to recharge in the Valley. In summary, urbanization has been accompanied by long-term stability in pumping and groundwater levels, plus the addition of imported SWP water to the Valley, which together have not reduced recharge to groundwater, nor depleted the amount of groundwater that is in storage within the Valley.

Effect of Pavement on Recharge Beneath Specific Land Parcels

The amount of paved cover on the ground affects the degree to which rainfall and outdoor-applied urban water will be able to infiltrate to groundwater. In heavily industrialized areas with high percentages of paved cover, such as exist in portions of the Los Angeles Basin, less rainfall recharge will occur than if the land is in an undeveloped condition. Furthermore, if the bottoms of rivers and other drainages are paved, then the majority of stormwater generated during a rainfall event will be unable to infiltrate to groundwater. In contrast, the amount of recharge to groundwater will be greater in urbanized areas, such as the Santa Clarita Valley, that have natural soils in the bottoms of rivers and local drainages or that have lower percentages of paved cover on the developed areas lying outside the principal drainages. In these areas, the outdoor use of water for irrigation landscape vegetation or agricultural lands can notably increase the amount of groundwater recharge, particularly if the outdoor water is imported from outside the local groundwater basin. This is discussed further below.

Effect of Vegetative Cover and Water Use

From the 1930s through the 1960s, H.F. Blaney and other researchers at the U.S. Department of Agriculture performed numerous studies to measure the amount of infiltration to groundwater that occurs beneath undeveloped lands and irrigated farmlands, and the differences in recharge rates for different types of native vegetation and crops. In California, these studies included a 1933 study by Blaney in Ventura County, a 1963 study by Blaney and others in the Lompoc Uplands, studies by the U.S. Geological Survey and various consultants in the Montecito and Carpenteria groundwater basins, and a groundwater basin study by Santa Barbara County[1] that incorporated the results of these earlier studies.

Together, these studies concluded that deep percolation to groundwater from undeveloped lands occurs only during years of average or above-average precipitation. This occurs because:

1. Southern California's rainfall is highly seasonal in nature, whereupon most rainfall occurs during the relatively cool period November through March, when plant water

[1] See Santa Barbara County Water Agency, December 15, 1977. *Report on Adequacy of the Groundwater Basins of Santa Barbara County.*

requirements are low, and little, if any, rainfall occurs during the remaining (and warmer months) when plant water requirements increase.

2. During the summer, when little or no rainfall occurs, the native vegetation extracts the residual moisture that is present in the soil, which substantially decreases the soil moisture within the root zone of the vegetation. At the end of the dry season, soil moisture levels on undeveloped lands are below the soil's field capacity, which is the amount of moisture that must be present in the soil before free drainage of water can occur below the rooting zone of the native vegetation.

3. When the seasonal rains arrive, the incident rainfall that is not consumed by plants and does not become stormwater runoff must first raise the soil moisture level to the soil's field capacity before any groundwater recharge will occur. The various studies indicate that about 17 inches/year of rainfall is necessary to raise the soil moisture to the field capacity on an undeveloped parcel of land. This is similar to the average annual rainfall in the Santa Clarita Valley and in other lowland coastal and near-coastal valleys in southern California.

On irrigated lands, irrigation occurs during several months of the year, with the exact duration depending on the amount and timing of rainfall and also the crops or type of urban landscaping being irrigated. The principal effect of converting undeveloped land to land that receives agricultural or urban irrigation is to increase the amount of water that is applied to the land during the low-rainfall months. This application of water to the vegetative cover on the surface of the developed land parcel results in the maintenance of higher soil moisture levels during the warm, dry months than would occur without development. This has three effects:

1. Because irrigation will generally be performed in a manner that maintains the health of the vegetative cover, enough water will be applied to maintain the soil moisture at, or close to, the field capacity of the soil. This in turn will allow some deep percolation to occur from the irrigation water itself.

2. When the rainy season begins, because irrigation has maintained soil moisture at or near field capacity, less of the initial rainfall entering the root zone needs to be stored in the soil (to meet soil moisture deficits) beneath an irrigated parcel than in the case of an undeveloped parcel. Therefore, a greater percentage of the initial rainfall and annual rainfall will be able to infiltrate to groundwater. The southern California studies estimated that irrigated land parcels would allow rainfall infiltration to occur in years when annual rainfall is at least 10.5 inches/year. This threshold rainfall value is 6.5 inches less than the threshold rainfall value that the studies estimated to be necessary for generating groundwater recharge beneath undeveloped land parcels.

3. Because the majority of irrigation occurs during the dry (low-rainfall) months, the total annual recharge to groundwater from irrigated developed lands is the sum of: (a) the deep percolation arising from irrigation (during the low-rainfall months); and (b) rainfall (during the months when less irrigation is occurring). Therefore, groundwater recharge beneath developed lands is greater and occurs for a longer period of time each year than in the case of undeveloped lands where no irrigation is occurring.

Historical Observations of Groundwater Conditions in the Santa Clarita Valley

The findings of the studies described above for other groundwater basins in southern California are consistent with observations that have been made in the Santa Clarita Valley, which are based on long-term water level records, water budget analyses, and groundwater modeling. Based on a month-by-month calibration to a 20-year record of historical water level records (throughout the Valley) and stream gaging records (at the Los Angeles – Ventura County line), the model simulates 10 percent of the applied outdoor water as being available for recharge to groundwater in retail and residential areas, with greater percentages infiltrating beneath golf courses and agricultural lands. This is consistent with a 1980 study by DWR of the groundwater resources of the Santee and El Monte hydrologic subareas of San Diego County. In that study, which was performed to evaluate reclaimed water use plans, DWR concluded that approximately 20 percent of the applied outdoor water in municipal areas infiltrates to the water table, with the remaining 80 percent going to evapotranspiration and direct evaporation. DWR also concluded that there would likely be no significant change in these percentages as urbanization continues.[2]

In the Santa Clarita Valley, as in any urbanized area, urbanization increases the paved area and can increase the magnitude and intensity of stormwater runoff from paved land areas. In the Santa Clarita Valley, this stormwater runoff will find its way to the Santa Clara River and its tributaries, whose channels are predominantly natural and consist of vegetation and coarse-grained sediments (rather than concrete). The stormwater that flows across paved lands in the Santa Clarita Valley is routed to stormwater detention basins and to the river channels, where the porous nature of the sands and gravels forming the streambeds allow for significant infiltration to occur to the underlying groundwater. Consequently, for a developed land parcel, the water that runs off of the paved portion of the land parcel will infiltrate to groundwater from a detention basin or a riverbed, rather than infiltrating onsite.

Riverbed infiltration is a significant percentage of total recharge in the Santa Clarita Valley in any given year. Streamflow records and the model calibration process together demonstrate that year-to-year fluctuations in total recharge in the Valley arise not just from year-to-year variations in incident rainfall within the Valley, but also from year-to-year variations in streamflows in the Santa Clara River and its tributaries. Because the areas contributing flow to the rivers are located both within and outside of the Valley, the recharge that occurs from riverbeds is a significant source of groundwater recharge within the Valley.

Evidence that stormwater infiltration to groundwater is not significantly decreased by urbanization comes from long-term water level records at wells completed in the Alluvial aquifer. These records show that groundwater levels and the amount of groundwater in storage were similar in both the late 1990s and the early 1980s, despite a significant increase in the urbanized area during these two decades. This long-term stability is attributed in part to the significant volume of natural recharge that occurs in the streambeds, which do not contain paved, urban land areas. Also, groundwater pumping volumes have not increased

[2] See State of California, Department of Water Resources, Southern District. August 1984. *San Diego County Cooperative Ground Water Studies: Reclaimed Water Use. Phase II.* Pages 40-41.

due to urbanization, compared with pumping volumes during the 1950s and 1960s when water was used primarily for agriculture. Additionally, beginning in 1980, water was imported into the Santa Clarita Valley from the State Water Project (SWP) for urban use, with SWP water use reaching nearly 30,000 acre-feet per year (AF/yr) by the end of the 1990s, and progressively increasing from about 32,500 AF in 2000 to nearly 44,500 AF in 2003. Because two-thirds of the total urban water demand is used outdoors, a substantial portion of the imported SWP water has been and continues to be applied to urban landscaping, thereby increasing the amount of recharge to groundwater. The remaining urban water is used indoors, and is subsequently routed to local water reclamation plants (WRPs) and then to the Santa Clara River (after treatment). A portion of this water flows downstream out of the basin, and a portion infiltrates to groundwater.

In summary, urbanization has been accompanied by long-term stability in pumping and groundwater levels, plus the addition of imported SWP water to the Valley, which together have not reduced recharge to groundwater, nor depleted the amount of groundwater that is in storage within the Valley.

APPENDIX F

ASSESSMENT OF POTENTIAL IMPACTS RESULTING FROM CUMULATIVE HYDROMODIFICATION EFFECTS, SELECTED REACHES OF THE SANTA CLARA RIVER, LOS ANGELES COUNTY, CALIFORNIA

Assessment of potential impacts resulting from cumulative hydromodification effects, selected reaches of the Santa Clara River, Los Angeles County, California

Report prepared for:

GeoSyntec Consultants

Prepared by:

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Assessment of potential impacts resulting from cumulative hydromodification effects, selected reaches of the Santa Clara River, Los Angeles County, California

Balance Project Assignment 205018

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October 27, 2005

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APPENDICES

Appendix A. List of aerial photographs analyzed

1. INTRODUCTION

1.1 Background and purpose

The Newhall Ranch Specific Plan projects will urbanize a portion of the Santa Clarita Valley in Los Angeles County during the coming decades. The project is an extension of prior community growth, which commenced in earnest during the 1960s, in accordance with the adopted General Plan and adopted growth projections. Concern has been expressed that future urbanization may result in changes in the Santa Clara River, a stream of regional scale draining westward from northern Los Angeles County through Ventura County, flowing into the Pacific Ocean near Oxnard. Prior analysis by Geosyntec Consultants (2005) indicates that cumulative future urbanization in the upper watershed of the Santa Clara River, of which Newhall ranch will contribute a portion, will reach approximately 9 percent at "built-out" conditions. A survey of the literature (reviewed in GeoSyntec, 2002) shows that many western-state streams begin to exhibit effects when impervious areas exceed a threshold of about 10 percent, with some considerable site-by-site variability. Additional studies by GeoSyntec in the San Francisco Bay area (2004) and a recent Southern California regional study (Coleman and others, 2005) indicate that, for watersheds smaller than about 25 square miles, channels in granular, non-cohesive sediments may become unstable downstream from urbanizing areas when impervious coverage reaches as little as 2 to 3 percent.

This report uses an empirical approach to assess the potential effects of urbanization on channel morphology associated with the implementation of the Newhall Ranch Specific Plan, combined with other existing and future development in the upper watershed of the Santa Clara River as described in the adopted General Plan. We use historical changes in the Santa Clara River channel pattern to help bracket potential morphological effects on the river of hydromodification due to accumulated urban development. We note that historical changes (both natural and human-induced) in the three factors most likely to affect the Santa Clara River stability (magnitude and frequency of stormflow events, sediment supply and caliber, and channel vegetation) are very large relative to the effects, if any, of the Newhall Ranch project and other planned future urban development. We hypothesize that it will prove useful to learn from history, and to assess the nature and general degree of change that may result from future urbanization by applying these insights.

Much of what is learned from this analysis may be applicable in other aspects of planning and managing the Santa Clara River in the Newhall Ranch reach and reaches downstream. It is not, however, an immediate objective of this report to develop management plans, to assess

potential changes in tributary channels, or to explore how habitat conditions might be changed by potential hydromodification, beyond that which is related to the physical channel form and dynamics.

1.2 Technical approach

The history of the Santa Clara River in the Santa Clarita Valley and eastern Ventura County allows us to explore the three factors most likely to affect the stability and morphology of the river downstream from existing and future development in the Santa Clarita Valley (including Newhall Ranch):

- High streamflows, including increased peak flows, volumes, and/or durations of stormflows,
- Coarse-sediment supply, including sharp curtailment of sediment entering the river following completion of Castaic (1974) and Santa Felicia-Piru (1958) Dams.
- Mature riparian vegetation, with interpenetrating roots, which can stabilize the banks and maintain the channel pattern.

We consider the 'pre-urban' condition to be the form and functions of the river during the 1950s and 1960s, prior to significant urban growth and modification of the flow and sediment regimes due to the construction of the Castaic and Santa Felicia-Piru Dams. Historic deviations from the pre-urban condition can be evaluated using the geomorphic evidence left by a period of floods and high flows from 1938 to about 1945. The effects of sediment supply can be evaluated by quantifying effects of eliminating coarse-sediment delivery from Castaic Creek (with a drainage area of 155 square miles, approximately 25 percent of the Santa Clara watershed at the L.A./Ventura County line. Supporting evidence can also be obtained similarly at Piru Creek (approximately 40 percent of the watershed at its confluence with the Santa Clara River at Piru).

1.3 Report organization

The analysis begins with an overview of the factors affecting the form and geomorphic history of the Santa Clara River (Chapter 2). The larger events and fluctuations, and manner in which they may have affected the river, are considered in Chapter 3. The fourth chapter explains the source materials and methods used to quantify the river's response to these perturbations, which are summarized in Chapter 5. Chapter 6 is a discussion of what we have learned from this study, and Chapter 7 draws conclusions as to how these findings relate to potential hydromodification effects in response to anticipated future watershed urbanization.

2. GEOMORPHIC SETTING

2.1 Channel pattern influences

Several previous reports have described the overall and geomorphic histories of the Santa Clara River (c.f., Schwarzberg and Moore, 1995; SCREMP 2005). In each case, authors have noted that the forms and functions of the river have varied with climatic cycles and with episodes such as floods and fires. It is this variability that is characteristic of the river. In the this report, we utilize the study of historic influences of some of the more pronounced events and cycles to better understand the impacts of drainage changes, if any, that can be expected to result from the anticipated future development in the Santa Clarita Valley, including Newhall Ranch.

2.1.1 Physiography

The Santa Clara River flows through a complex, tectonically-active trough generally bounded by reverse faults on the San Cayetano Mountain and South Mountain fronts. Some of the most rapid rates of geologically-current uplift in the world are reported from the Ventura anticline and San Gabriel Mountains, just to the northwest and southeast, respectively, of the river. Slopes are very steep, with local relief of 3000 to 4000 feet being common. These faults bring harder, more resistant sedimentary rocks over softer and younger sedimentary formations, but all formations are fundamentally soft and erodible. On either side of the faults, sandstone (generally multi-cyclic and fine-grained) and mudstones prevail. The northeastern and southeastern corners of the watershed are underlain by deeply-weathered granitic and schistose rocks, which produce sands that are coarser than those of other rock units when they weather and erode. The San Gabriel fault crosses the valley near the county line, bringing slightly more resistant rock to the surface and creating a local base level reflected as a slight rise or 'bump' on the river's longitudinal profile.

Most geologic materials in the watershed decompose mainly to silts and clays and to sand, with some coarser materials. Rhea Williams and his colleagues at the U. S. Geological Survey found that most sediment moved by the Santa Clara River and its main tributaries are quite fine, with less than 5 percent bedload-sized material (>0.25 mm, or about 0.01 inches in diameter). Some gravels and cobbles do occur within the beds of the streams and in their alluvium. Nonetheless, both the bed and the sediment transported by the river tend to be finer than in most Southern California watersheds (c.f., Knudsen and others, 1992).

The Santa Clara River watershed drains a watershed of 1,600 square miles, of which 625 square miles are within Los Angeles County, upstream of the "county-line gage" (USGS No. 11108500), near the western edge of the Newhall Ranch Specific Plan area.

2.1.2 Climate

Much of the watershed upstream of the Newhall Ranch Specific Plan area receives rainfall averaging about 18 to 25 inches per year (NOAA). As throughout Southern California, rainfall in the Santa Clara watershed alternates between wet and dry periods, a variation that is central to understanding the cultural and geomorphic histories of the upper watershed (Schwarzberg and Moore, 1995; Lynch, 1931; Reichard, 1981). Wet cycles tend to persist for several years, sometimes for periods of 6 or 8 years, during which rainfall, although variable, may average about 140 to 150 percent of the long-term average. For the woody riparian vegetation along the banks and on islands in the braided channels, these are crucial periods for establishment and growth. During dry cycles, the roots of the riparian vegetation must grow downward to the water table or perched zones, and where it cannot do so, this band of vegetation will die back.

2.1.3 Flows

Flows in the Santa Clara River, as in most southern California streams, are highly episodic. For the gaged period between 1953 and 1996 annual flow at the Los Angeles/Ventura County line gage ranged between 253,000 acre-feet (1969) and 561 acre-feet (1961). In general, however, streamflow, and especially dry-season streamflow, has increased over the past few decades primarily due to discharges from two wastewater treatment plants. Mean annual flow at the County Line increased from 25,700 acre-feet in 1972 (averaged over a 20-year record) to 35,360 acre-feet in 1988 (36-year record), with a significant decrease in the number of very low years over that period (UWCD and CLWA, 1996). Downstream of the County line, however, the Santa Clara River flows through the Piru groundwater basin, which represents a "Dry Gap" where dry-season streamflow is lost to groundwater.

Annual peak flows at the County line between 1953 and 1996 ranged from 68,800 cfs (1969) to 109 cfs (1960). Of note is that the second highest annual peak, 32,000 cfs in 1966, was less than half of the highest peak (68,800 in 1969). Both of these events occurred in the late pre-urban to early-urbanization stages within the Santa Clarita Basin and no consistent increase in peak flow is evidence since this time. Flow data for the 2005 flood event are not yet available, however the peak flow at the County line may have approached the flow observed in 1969. As discussed below these large episodic events have a significant impact on the geomorphic characteristics of the Santa Clara River mainstem.

2.1.4 Ground-water supported riparian vegetation

The Santa Clara River is underlain by several distinct alluvial ground-water basins—the Piru, Fillmore, and Santa Paula Basins (Reichard and others, 1999; SCREMP 2005). These basins are divided longitudinally by sills or ridges of bedrock that support areas of locally-high ground water, including the area upstream from the County line (above the Piru Basin), and upstream from the mouth Sespe Creek (the transition between the Piru and Fillmore Basins). This locally-high ground water sustains summer baseflow and riparian vegetation within the Santa Clara River corridor even through relatively dry climatic cycles.

3. PERTURBATIONS

This section describes several major perturbations (those with the potential to affect channel- and floodplain-form) that occurred in the Santa Clara River watershed since the early 1900s (summarized in Figure 1). Aerial photographs were selected to bracket these events and analyzed, both qualitatively and quantitatively, to try to discern and quantify responses of the Santa Clara River channel to:

(1) changes in flow regime during wet and dry multi-year cycles,

(2) sediment supply, notably describing the channel's adjustments to construction of large dams, and

(3) development of mature riparian vegetation with interpenetrating roots.

3.1 Streamflow cycles and events

As described above, streamflow within the Santa Clara watershed is highly episodic, and can vary drastically from year to year. However, decade-scale patterns of wet and dry periods have been identified in the historic record—as early as the 1700s. Previous wet periods (with associated high flows) are reported from 1810 to 1817, 1831 to 1840, 1883 and 1893, and 1903 to 1916, during each of which periods the area received a total of an additional 60 to 80 inches above the mean annual rainfall over the duration of the wet cycle. Prolonged static or drying periods similar to that observed between 1945 and 1977 also occurred from 1780 to 1810, 1842 to 1882, and 1919 to 1935 (with associated reductions in streamflow). The river is likely to have remained most stable during the latter periods, with the notable exceptions of a few major storms of record, such as 1862 (c.f., Lynch, 1931; Reichard, 1981; Schwartzberg and Moore, 1995). The primary wet periods in this study occurred between 1938 and 1946, and 1978 to 1983 (Figures 1 and 2). Other large storm events occurred in 1966, 1969, 1972, 1983, 1998, and 2005. Notable dry periods occurred between 1946 and the late 1960s, and 1983 and 1991.

3.2 Dam construction

Castaic Dam was completed on Castaic Creek (a tributary of the Santa Clara River just upstream of the Newhall project) in 1974. The watershed area above the dam is approximately one-quarter of the watershed area of the Santa Clara River at the L.A./Ventura County line, downstream of the Castaic confluence, and therefore the dam effectively reduced the sediment contributing area by about 25 percent. For comparison purposes, we also considered the effects

of the construction of the Santa Felicia Dam (Lake Piru), which resulted in an approximate 38 percent decrease in sediment contribution area below the confluence of Piru Creek and the Santa Clara River[1].

3.3 Urbanization

Settlement of the Los Angeles County portion of the watershed transitioned from rural to mixed-use suburban during the mid- to late-1960s. This change initiated a period of ongoing urban expansion, with associated increases in the area of impervious or compacted surfaces as homes, commercial and industrial centers, highways and diverse infrastructure have developed throughout the Santa Clarita Valley. Future General Plan urbanization within the upper watershed, inclusive of Newhall Ranch, will bring the percent of urban area west of the County line to about nine percent (GeoSyntec, 2005).

3.4 Treated effluent discharge

Since the 1960's, treated effluent from two water reclamation plants (Saugas and Valencia) has been released directly to the Santa Clara River. This, combined with an increase in applied, imported agricultural water, has led to increased summer baseflows in the Santa Clara River at the County line, which had only rarely occurred under pre-urban conditions. This led to an increase in available water to support woody riparian vegetation. The increase in baseflow is evident in the USGS gaging record at the county line (Figure 2). In some stream corridors, vegetation growth in response to increased baseflow can provide additional bank cohesiveness and reduce erosion; though in others heavy in-channel vegetation growth (riparian encroachment) can serve to destabilize the stream and induce lateral erosion by directing flows toward the banks.

Newhall Ranch has proposed an additional plant that would ultimately treat approximately 5.8 million gallons per day at project build-out. However discharge from the plant in the summer is not expected, as this water will be re-used for irrigation purposes, and we therefore do not expect further change in riparian vegetation growth as a result.

3.5 Saint Francis Dam Breach

On March 12, 1928 the Saint Francis Dam, located in San Francisquito Canyon upstream of the Newhall project, failed and released approximately 30,000 acre-feet of water over the course of a few hours, with an estimated peak discharge of up to 800,000 cubic feet per second (Newhall,

[1] Drainage area calculations were based on USGS gaging station watershed data at Piru and Castaic Dams, and gages on the Santa Clara River at the L.A./Ventura County line and near Piru.

1928; and SCREMP, 2005). This event had drastic effects on the stream reaches downstream, as the resulting flows were much higher than anticipated from any natural event. Aerial photograph coverage during this time period is limited, however, and therefore an assessment of this event was not feasible. In addition, because of the extreme size of the event, it is unlikely that an assessment would be beneficial for assessing hydromodification impacts.

4. METHODS

We analyzed aerial photographs from 1927, 1947, 1957, 1966/67, 1989, 2002, and 2005 to describe channel change in response to the major episodes described above. The main criteria described were the width of the active braiding area (or meander belt width if there was no braiding), bank vegetation, number of channels, and width of the active channel. Also described, where they could be identified, were the width and length of "islands" (vegetated mid-channel bars) within the stream. Islands were typically easier to identify where vegetation was heavy, as the color of the vegetation highlighted the differences between channel and meta-stable islands.

The aerial photographs were analyzed in two different ways. First, a qualitative comparison of the alluvial corridor shown in the different years' photos was made, describing general differences in channel pattern and vegetation on a reach-wide scale. Second, specific cross sections were defined and the above parameters measured for each year with photo coverage in that area to provide a quantitative comparison of channel change at these standard locations along the Santa Clara River (Figure 3).

4.1 Descriptions of analysis criteria

4.1.1 Width of active braiding corridor

For braided reaches, the active channel width was identified primarily by noting the extent of active channels or recent sediment deposition. In many cases the active corridor was bounded by a significant change in vegetation or sediment deposition characteristics.

4.1.2 Relict channel corridor

The relict channel corridor is the portion of the flood plain that does not appear to have been active in the recent past (within the last 5 years or so). Typically the relict corridor is identified by areas of heavy or scattered vegetation containing no or few distinct channels, or areas that do not appear to have experienced recent sediment deposition. Alternatively, identification was based on the width between farmed fields[2]. Measurements of this feature were made from outside bank to outside bank, and include the active corridor.

[2] The total width of the former channel migration corridor is difficult to identify in aerial photographs due to past and present agricultural field reclamation following major perturbations. Where necessary, we used the width between agricultural fields as a estimate of the relict corridor.

4.1.3 Channel width

Where a distinct channel or channels could be identified, the widths of the individual channels were measured. The number of individual channel threads was also recorded, where threads could be distinguished. In some cases, measurement of these features was complicated by poor photo resolution or contrast, and difficulty in distinguishing major channels from minor ones (where a full spectrum was present).

4.1.4 Vegetation

Vegetation was described qualitatively as bare, scattered, moderate, and heavy. The location of specific areas of vegetation, such as vegetated islands, vegetation within the relict corridor, or vegetation along banks, was also described. Where the resolution was adequate, the growth form of vegetation, or state of maturity, was also described (trees or shrubs).

4.1.5 Number of vegetated islands

The number of distinct vegetated islands (mid-channel bars) was also recorded at each cross-section, where the resolution of the photographs was adequate. Where islands could be identified, measurements of width and length were recorded.

5. RESULTS

5.1 Qualitative descriptions

Initial inspection of the series of aerial photographs showed that significant changes in channel planform have occurred throughout the 1900s, as would be expected in a large, braided stream in southern California. Vegetation within the relict corridor (see definition above) near the Newhall Ranch planning area appears to become progressively heavier through time, likely due to the increase in agricultural water and discharge of treated effluent to the channel through the summer months.

The photos show many areas of net deposition, and corresponding channel shifts in major depositional areas. Single-thread, dominant channel segments are rarely present, especially in years following large events. Even when there is one main channel, secondary channels are often present within the active channel corridor.

Portions of the stream have been altered for flood control purposes, including stabilization of banks bounded by orchards and fields, or construction of levees within the active corridor. These levees are most prominent in the 1989 photographs (upstream of the L.A./Ventura County line), where the substantial segments of the main channel are confined in a flood control channel approximately 225 feet wide. By 2002, however, little evidence can be discerned in the aerial photographs of these levees.

The 2005 flood events caused significant changes within the Santa Clara River. Vegetation within the channel was almost all completely washed out (compared to 2002 conditions), and many areas of significant bank-widening were identified, even in areas of heavy bank vegetation (Figure 4).

There appears to be little change in agricultural constriction of the Santa Clara River over the span of photographs reviewed. Through the Newhall reach, the agricultural areas appear to be well buffered by the relict channel and the vegetation supported there. There were only a few places identified where the active channel cut into agricultural areas rather than staying within the relict corridor. In contrast, within the Piru Basin (downstream of the Newhall reach), significant agricultural constriction and subsequent channel widening occurred over the time span of the photos reviewed.

Areas of shallow ground water between Piru and Sespe Canyon[3], which support denser riparian vegetation than typical for the river between Valencia and Fillmore, show little if any significant change for all years in the studied photo-sets. Both the density and extent of vegetation in these areas does not appear to change over time (despite significant differences in climate and other watershed factors) nor does the amount of vegetation appear to significantly affect channel planform, compared to upstream and downstream reaches (the braided channel does not shift to a single-threaded channel through the wetted reach).

5.2 Quantitative results

For the quantitative portion of the aerial photograph analysis we looked at four different types of criteria to identify physical changes to the Santa Clara River channel (Table 1; see also section 4.1.1 for descriptions of criteria). Because of difficulties in identifying and measuring the width/number of channels and number/dimensions of vegetated islands, because of the varying resolutions and contrasts of the photographs, we concluded that analysis of these two criteria were less meaningful for this study. In other words, there was more variation due to the ability to identify the features for the varying quality of the photos than there was actual variation in the system. While we believe that these criteria may be a valid indicator of channel change, more study would be needed to adequately quantify these features so they were used a supplementary qualitative metric.

For this study we found that measurement of the "active corridor" (see section 4.1.1) was the most useful and easiest to work with to identify channel changes. In most cases there is enough vegetation along the banks that the active braiding corridor is easily identified, and changes in the width of the corridor can be tracked from year-to-year.

Figure 5 summarizes the changes in active corridor width over the time span of the reviewed photos. Within the Newhall reach, the width of the "active corridor" at the four measured cross-sections varies from year-to-year by as much as 500 feet, though most of the variation is considerably less. One station, in the narrows above the Piru Basin, has a very consistent channel width, varying by less than about 50 feet from year to year.

To provide additional analysis, we looked at a series of recent photos (1994, 2000, and 2002-2005) at one cross section downstream of the Castaic confluence. For this photo set, the channel widened significantly between 1994 and 2000 (probably in response to the 1995 or 1998 large

[3] See Reichard and others (1999) for a discussion of the hydrogeology of these shallow ground water areas; although downstream from the Los Angeles County line, results are applicable to the upstream as well, as discussed later in this report.

storms), but showed almost no change between 2000 and 2004 (Figure 6). The channel then widened considerably again in response to the high-flow events in 2005.

As a secondary check of the numbers derived for the measured standardized cross sections, we also measured active channel widths at approximately twenty different locations through the Newhall Reach on three different photo sets—1967, 2004, and 2005. From these measurements an average active braiding corridor width was calculated and compared with the other years. In 1967, the average channel width was approximately 580 feet, which was significantly wider than the average width in 2002 (392 feet). However, after the 2005 storms, the active width was approximately 560 feet, similar to the 1967 conditions.

The "relict corridor" (see section 4.1.2 for definition) also proved useful as a secondary criterion, providing a measurement of potential changes due to agricultural encroachment or constriction of the flood corridor. Measurement of the "relict corridor" at the standard cross sections showed that while there was some variation between photos, there is no consistent trend of agricultural constriction to the Santa Clara River flood corridor. These measurements, along with qualitative observations that within the Newhall reach agricultural activities were generally restricted to outside the active corridor, suggest that agricultural encroachment has not historically affected the geomorphology of the Santa Clara River within the Newhall Reach.

6. DISCUSSION

The Santa Clara River is a dynamic, episodic system. The above analyses highlight the magnitude of geomorphic change over the course of recent history, in response to natural and human disturbances in the watershed. Understanding the magnitude of past response is a key factor in assessing the potential response to future urbanization within the watershed.

The construction of Castaic Dam in 1974, regulating approximately 25 percent of the watershed at the L.A./Ventura County line, cut off a significant supply of sediment to the Santa Clara River. This change, however, does not appear to have had an effect on the channel dimensions of the Santa Clara River mainstem. The width of the active corridor, as well as the general form of the channel, are generally consistent both before and after construction of the dam. It appears that the Santa Clara River adjusted without morphological expression to absorb this change. One factor contributing to the lack of change is the seemingly large volume of sediment stored in the tectonic basin above the county line—a result of bedrock control associated with movement along the San Gabriel fault, which supports the large extent of semi-consolidated and alluvial deposits adjoining the drainage net.

The amount of vegetation within the Santa Clara River corridor appears to have increased since the 1960s, likely due to the increased summer return flows from agricultural water and to year-round augmentation of baseflows due to treated effluent discharge to the river. However, this vegetation does not seem to provide enough erosion resistance to maintain a "stable" channel capable of withstanding regular 're-sets', which occur at intervals averaging about a decade – or much less than the expected lifetime of the riparian woodlands which do get established. Despite heavy vegetation on the active channel banks near Newhall ranch and in areas of shallow ground-water, the stream still responds to large events by a general widening and/or shift of the channel. The role of vegetation in large-channel stability and morphology in Southern and Central California does fundamentally differ from that of smaller streams and streams elsewhere in the country. The geomorophic and historical record shows that resets have been occurring throughout the recent geologic past in basins exceeding a certain size. One partial explanation may be that 're-set' flood events in these larger channels exert stresses beneath or around the riparian vegetation exceeding the vegetation's threshold of stability[4].

[4] Sedimentologists note that crossbeds in the alluvium of the Santa Clara River are often 8 to 12 feet high, equal or greater than the depth to which roots can interpenetrate in most riparian settings in the region.

As stated above, the Santa Clara River, as with many streams in semi-arid southern California, is highly episodic. Concepts of "normal" or "average" sediment-supply and flow conditions have limited value in this "flashy" environment where episodic storm and wildfire events have enormous influence on sediment and stormflow conditions. Many of these channels are actively adjusting to lower flows than the last major event, which may have occurred some years before[5] (Hecht, 1993). In these streams, a large portion of the sediment movement events can occur in a matter of hours or days. In many of these channels most sediment is moved—and most bed changes occur—during the large flow events resulting from storms that may be expected approximately every 5 to 15 years (c.f., Capelli and Keller, 1993; Hecht,1993; Inman and Jenkins, 1999; Knudsen and others, 1992; Kroll and Porterfield, 1969).

Evidence of episodic channel changes can be seen in the Newhall reach of the Santa Clara River. Based on aerial-photograph interpretation of a near-yearly sequence of aerial photographs from within the last decade, the channel appears to maintain a consistent planform during average or dry rainfall years (such as between 2000 and 2004). Large events, however, (such as that which occurred in February 1998 and January 2005) can significantly modify this channel form. This widened and/or shifted channel (like that which was present after the 1998 or 2005 stormflow events) then sets the geomorphic template for subsequent normal to dry years. This model, similar to that described for the Ventura River by Capelli and Keller (1993), suggests that the geomorphology of the Santa Clara River is primarily driven by these large events.

Other perturbations which potentially affect channel geometry appear to have transitory or minor manifestations. For example, effects on the channel width due to 1980s levee construction are barely discernible by the first few years of the 21st century, probably mostly due to morphologic compensation associated with the mid- to late-1990s storm events.

[5] Actively adjusting channels may be aggrading, incising, expanding or otherwise changing channel dimensions, depending on the magnitude, type, and various effects of the episodic event.

7. CONCLUSIONS

Based on the study of historic aerial photographs described above we conclude that:

- Major perturbations within the Santa Clara River watershed (dam construction, levee construction, changes in flows in response to decadal-scale climatic patterns, and increases in woody vegetation) do not appear to have had a significant impact on the geomorphic expression of the Santa Clara River, as quantified from measurements made from a series of historical aerial photographs flown during the years 1927 through 2005.

- Large events (those which are typically not as affected by increases in impervious area and associated increases in stormwater peaks and runoff volume) can completely alter the form of the Santa Clara River channel. We call these events "re-set" events. These events, perhaps occurring on average once every ten years, are a dominant force in defining channel characteristics.

- The geomorphic dominance of "re-set" events overwhelms geomorphic effects of hydromodification on smaller events. Due to these episodic "re-sets" we do not expect hydromodification feedback "unraveling" of the Santa Clara River mainstem, as is seen in many smaller southern California watersheds[6]. The "re-set" events appear to adequately buffer changes that may occur in short-term sediment transport.

- While there is no expected increase in summer flows due to additional treated effluent discharge to the Santa Clara River, even if summer baseflow do increase we would not expect a significant change within the channel. Additional growth in the extent or density of vegetation is not anticipated, as the reach near Newhall already appears to have enough flow to support summer vegetation, and the existing vegetation does not appear to affect channel form for durations longer than the "re-set" interval. Further, re-sets occur at intervals significantly shorter than the period required for maturation of riparian vegetation, such that full development of bank-holding properties is frequently interrupted.

- Given that the channel morphology of the Santa Clara River mainstem has not adjusted significantly to much larger perturbations in flow, sediment yield, and riparian

[6] In many smaller streams, hydromodification of moderate events can induce incision of the stream bed, which reduces the connection of the stream to the floodplain. This disconnect, in turn, increases the erosive forces of the flows (concentrating more flow in the channel) and causing further erosion, and thus a positive feedback response.

vegetation growth factors, within the Newhall reach, we do not expect a significant geomorphic impact to the Santa Clara River mainstem due to the anticipated increase in 'urban area' from four to nine percent.

8. LIMITATIONS

The analyses in this report were designed to help bracket the range of likely effects on the geomorphology of the Santa Clara River due to proposed urban expansion under the General Plan, inclusive of the Newhall Ranch Specific Plan projects. It does not consider specific elements of the project or of evolving mitigation measures; rather, it focuses upon the susceptibility to perturbation of the Santa Clara River corridor as a whole. We believe that it conforms with the standard of care applicable to reconnaissance studies of this nature; no other warranty, expressed or implied, is made.

The above analyses and discussion were intended to assess the potential cumulative impacts to the Santa Clara River *mainstem* (not tributaries) due to the anticipated urban expansion in the watershed. While we conclude that urban expansion from approximately four- to nine-percent urbanized (not 'impervious') will not significantly affect the channel geomorphology of the Santa Clara River, we do expect that there might be a response to urbanization on a larger scale. However, further study would be required to define what the likely threshold and magnitude of response might be.

We ask readers to note that this is a reconnaissance report. It is intended to bracket likely future conditions, to identify factors which must be better known, and to help guide initial planning. This report should *not* be used to site or design individual facilities without further site-specific investigations. Similarly, it is *not* intended to serve as basis for flood management or detailed floodplain planning, both of which should be conducted by well-defined and site-specific procedures, and which frequently require multiple lines of evidence.

The application of geomorphic history to inferring future channel and corridor change has a long and respected record in the earth sciences. As with all history or archival analysis, the better the record is known and understood, the more relevant and predictive the analysis can be. We do encourage readers who have knowledge of other events or processes which may have affected the river to let the authors know at the first available opportunity. The authors and their contacts via several different media are given on the signature page of this report.

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TABLES

Table 1. Aerial photograph cross section data at selected locations near Newhall Ranch, Los Angeles County, CA. See text for explanation and interpretation of data. Locations of cross section are labeled on Figure 2. Photo sources are listed in Appendix A.

Cross section	location description	photo date	width of active braiding corridor (feet)	width of relict braiding corridor (feet)	is there one primary channel visible?	width of main channel (feet)	number of identifiable channels	total width of channels (including main) (feet)	number of islands	length of islands encountered (feet)	width of islands (feet)	vegetation	other descriptions
X1	downstream of Castaic	8/16/1947	570	1247	yes?	71	3?	107	can't define	n/a	n/a	moderately vegetated with some portions of relict corridor heavily vegetated	Just downstream a heavily vegetated bar is cut by a very distinct secondary channel
		7/20/1966	729	1173	yes	27	1	27	1	497	86	almost no vegetation within primary corridor except two areas near the primary channel and scattered small patches, only scattered vegetation on relict corridor	while there is only one main channel the rest of the primary corridor is section is almost deltaic in planform, spreading out from constriction upstream (possibly high sediment load coming in from Castaic)
		5/26/1989	173	1171	yes, but small	43	1	43	0	n/a	n/a	banks of meander corridor have scattered vegetation (less than 2000) with very little within braiding corridor	meander corridor is very distinct and straight, could be from flood control dredging;
		6/1/1994	337	1167	yes	72	2	97	1	551	171	light to moderate vegetation on braiding corridor banks	very little vegetation within braiding corridor
		2/1/2002	505	984	yes	42	2	50	poorly defined	n/a	n/a	relict braiding corridor is well-vegetated; meander belt/bar is lightly to moderately vegetated; at least one main channel bank is well-vegetated (alternates w/ meanders)	secondary channel essentially cuts off meander
		4/1/2004	505	978	no	n/a	3	87	2	929, 251	248, 56	heavy vegetation along former primary channel; relict corridor also heavily vegetated	there are two distinct channels, approximately the same size
		3/1/2003	510	965	yes	75	1	45	0	n/a	n/a	heavy vegetation on northern bank; some scattered vegetation within active corridor and surrounding low-flow channel	channel branches just downstream of cross section; very similar to 2002 and 2004 photos
		2/1/2005	601	999	no	n/a	3	106	poorly defined	n/a	n/a	no vegetation in main portion of channel; right bank has heavy tree cover, left bank has few trees	the main channel is about 340 feet wide with an obvious overbank deposition area (with very little vegetation)
X2	Upstream of County line	8/16/1947	532	1197	yes	89	2	133	1	355	133	vegetation is heavy (probably trees) on relict corridor; moderate (probably scrub) within active corridor (difficult to distinguish)	very distinguishable difference between active and relict corridor within this reach
		3/6/1963	491	1352	no	n/a	difficult to define	n/a	6	252, 283, 82, 441, 94, 410	44, 57, 52, 76, 38,63	several well-defined islands behind established vegetation (individual shrubs or small trees); relict corridor has moderate to heavy tree cover	very braided planform; switches to predominately single-thread channel just downstream
		5/26/1989	651	651	yes	43	3	108	1	2385	477	relict corridor has scattered trees with moderate to heavy shrub or grass cover; central island (along levee) has similar vegetation	well-defined flood control channel, but has been breached and there is a significant secondary channel to the north of the levees; included a portion of the island between the flood control channel and the secondary channel in the relict channel (no sign of recent deposition)

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Cross section	location description	photo date	width of active braiding corridor (feet)	width of relict braiding corridor (feet)	is there one primary channel visible?	width of main channel (feet)	number of identifiable channels	total width of channels (including main) (feet)	number of islands	length of islands encountered (feet)	width of islands (feet)	vegetation	other descriptions
		6/1/2002	608	1258	yes	131	1	131	0	n/a	n/a	relict corridor on north bank has heavy tree cover; meander bends are eroding tree bank vegetation in places	stream has meandering planform, though meander belt (400' wide) has high sediment deposition and little vegetation; no evidence of flood control levees (meanders have widened to erode levees); active channel includes meander belt and area of significant recent sediment deposition to the north of the meander belt
		2/1/2005	674	1240	yes	97	3	192	1	475	155	almost no vegetation within active channel; relict corridor on both banks has moderate tree cover; much vegetation eroded away since 2002	numerous very small channels present as well
X3	downstream of county line	8/16/1947	362	805	yes, at this xs	80	2	121	can't define	n/a	n/a	outer banks of braiding corridor seem heavily vegetated	there seems to be one main channel through this reach, with extensive deposition of sediment outside of the channel
		7/20/1966	140	714	yes	51	2	77	0	n/a	n/a	banks of braiding corridor are heavily vegetated	
		5/26/1989	273	864	yes	91	2	114	1	136	23	only scattered vegetation on banks of braiding corridor	braiding corridor looks as though it may be a leveed flood control channel
		2/1/2002	249	1466	yes	41	3	79	2	344, 219	66, 36	scattered vegetation on u/s ends of islands; some recent deposition of sediment within relict braiding corridor (which is predominately heavily vegetated	
		2/1/2005	587	1472	yes	97	3	145	1	543	110	no vegetation in active corridor; right bank has heavy shrub cover with some trees, left bank has light shrub cover	
X4	upstream of Piru Basin	8/16/1947	282	885	yes	121	1	121	can't define	n/a	n/a	little to no vegetation within braiding corridor; relict braiding corridor has heavy tree/shrub cover	
		7/20/1966	281	383	no	n/a	3	26	poorly defined	n/a	n/a		
		5/26/1989	318	591	yes	68	1	68	1	91	23	meander belt banks lined with trees; meander belt itself covered with shrubs	"braiding corridor" is actually the meander belt; meander belt outside of channel is heavily vegetated
		2/1/2002	266	426	yes	35	3	45	1	340	36		secondary channels may be present in other photos, but resolution is poor, esp. 1948
		2/1/2005	281	495	yes	44	1	44	0	n/a	n/a	vegetation on right bank of main channel has diverted some flow over the relict corridor, though conditions are similar in 2002; moderate to heavy trees and shrubs on both banks	conditions are very similar to 2002, but with slightly wider and much clearer channel

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Cross section	location description	photo date	width of active braiding corridor (feet)	width of relict braiding corridor (feet)	is there one primary channel visible?	width of main channel (feet)	number of identifiable channels	total width of channels (including main) (feet)	number of islands	length of islands encountered (feet)	width of islands (feet)	vegetation	other descriptions
X5	upstream of Piru confluence	4/1/1927	1834	3191	no	n/a	many	n/a	3	3060, 1170, 468	540, 450, 90	sparse scrub vegetation within active corridor, but enough to define the complex channel pattern; only slightly more vegetation (or possibly just less recent sediment deposition) in relict corridor	relict channel is mainly an artifact of flow deflection by several long levees just upstream; typical braided stream with channels of varying widths and scales (can not define number of channels due to complexity and scale variation of channels); only measured large islands
		8/16/1947	1449	3066	no	n/a	0	n/a	1	1282	279	island appears heavily vegetated; relict channel has moderate vegetation, possibly some farming	active channel is very burnt in; no evidence of levees, but would be difficult to see
		11/10/1966	957	3051	no	n/a	complex channel pattern	n/a	too complex to define	n/a	n/a	no vegetation within active corridor; sparse scrub vegetation within relict corridor, but very patchy (may be due to clearing)	flood control channel is present down middle of active corridor (196' wide); stream has complex braiding pattern, even with flood control channel present
		6/20/1989	1796	2993	no	n/a	complex channel pattern	n/a	too complex to define	n/a	n/a	light scrub vegetation within active corridor; vegetation is obviously stabilizing small islands, at least until the next big event; relict corridor is sparsely vegetated	little evidence of flood control channel but may have been some excavation in middle of active corridor (~300' wide);
		6/1/2002	1730	2452	no	n/a	5	1000	3	1200, 1085, 1520	384, 406, 400	moderate scrub vegetation on islands within active channel, similar to 1989 but slightly heavier	channels were relatively easy to pick out due to moderate scrub vegetation; channel width does not necessarily correlate to other measurements (where the only measurable parameter was wetted width)
X6	downstream of Piru confluence	4/1/1927	1713	1983	yes	18	1	18	0	n/a	n/a	no vegetation within braiding corridor; only scattered vegetation on relict corridor; heavy trees along portions of the south bank of relict corridor	very wide braided corridor with little definition (too burnt-in to define secondary channels)
		8/16/1947	1767	1983	no	n/a	0	n/a	0	n/a	n/a	looks similar to 1927 conditions	
		9/1/1957	1220	1449	yes	25	3	51	2	875, 1750	325, 425	very sparse scrub vegetation in active corridor; some small trees on relict corridor (where corridor is present)	well-defined flood control channel through this reach (136' wide), but there are several secondary channels outside the levees; diversion ponds present near the north bank; larger island cut by flood control channel
		11/10/1966	1132	1563	yes	32	4	388	2	2125, 750	850, 250	large island is moderately vegetated with scrub and one line of heavy vegetation; relict braiding corridor is similarly vegetated	braiding corridor has been confined on both sides by levees (especially on the northern portion); looks like the southern levee was recently overtopped (that area was included in the relict corridor); main channel divides in two in some areas
		6/20/1989	1082	1082	no	n/a	n/a	n/a	1	685	180	sparse scrub vegetation growing on poorly-defined islands within channel and near piers	lots of recent grading within the channel, several levees in the middle of the corridor and a series of piers on the southern bank
		6/1/2002	1050	1245	no	n/a	none	n/a	0	n/a	n/a	very little vegetation in this portion of the stream; some scattered scrub on relict corridor, even less within active channel	217-foot wide flood control channel begins just d/s of xs (poorly defined, though)

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Cross section	location description	photo date	width of active braiding corridor (feet)	width of relict braiding corridor (feet)	is there one primary channel visible?	width of main channel (feet)	number of identifiable channels	total width of channels (including main) (feet)	number of islands	length of islands encountered (feet)	width of islands (feet)	vegetation	other descriptions
X7	between Piru and Sespe (ground-water upwelling)	8/16/1947	1694	2472	no	n/a	4	difficult to define the widths	can't define	n/a	n/a	this area is heavily vegetated; difficult to distinguish active braiding corridor from relict corridor	looks like there has been some flood control work in this area, two very straight channels through here, but masked some by vegetation
		9/1/1957	1446	2253	yes	168	4	370	2	4624, 8500	272, 408	northern portion of the corridor (including flood control channels) have heavy vegetation outside of the channels; the southern portion of the corridor has sparse vegetation	the main channel, and possibly the secondary channel, have been altered for flood control
		6/20/1989	749	2697	yes	37	2	150	1	1386	449	thick vegetation (with trees) along main channel; very little vegetation otherwise within active braiding corridor; moderate vegetation in northern portion of relict corridor, but only scattered brush in southern	no evidence of flood control alteration; downstream the corridor has been severely constrained by encroaching agriculture
		6/1/2002	551	2767	yes	42	2	65	1	396	108	heavy vegetation (trees) along secondary channel along north bank; scattered shrub (with some trees) vegetation within active corridor, some defining the edges of bars; heavy scrub vegetation on south relict corridor with scattered trees; heavy trees and scrub on northern relict corridor	just upstream there is a distinct main active corridor and an overbank area of deposition; the main active corridor has portions lined with heavy trees, but becomes less distinct further upstream (no vegetation)
X8	just downstream of Sespe Creek	8/20/1947	2003	2003	no	n/a	6	601	can't define	n/a	n/a	limited, if any	photo very burnt in, but channels less well-defined than in other photos
		8/13/1967	701	2203	yes	100	3	250	1	2804	401	limited, if any	one single-thread channel with one minor channel
		6/20/1989	1532	1723	yes, but less so than 1967	153	5	306	poorly defined; small and well-vegetated	n/a	n/a	islands are more heavily vegetated away from main channel; main channel bank is ~75 vegetated w/ thin vegetation line; more vegetation than in other photos	
		6/1/2002	670	1820	no	n/a	3	170	1	801	216	islands are moderately well-vegetated; relict corridor has scattered vegetation, Sespe mainstem has heavy vegetation along low-flow channels	interpretation complicated by Sespe confluence, but looks very similar to 1989 photo

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FIGURES

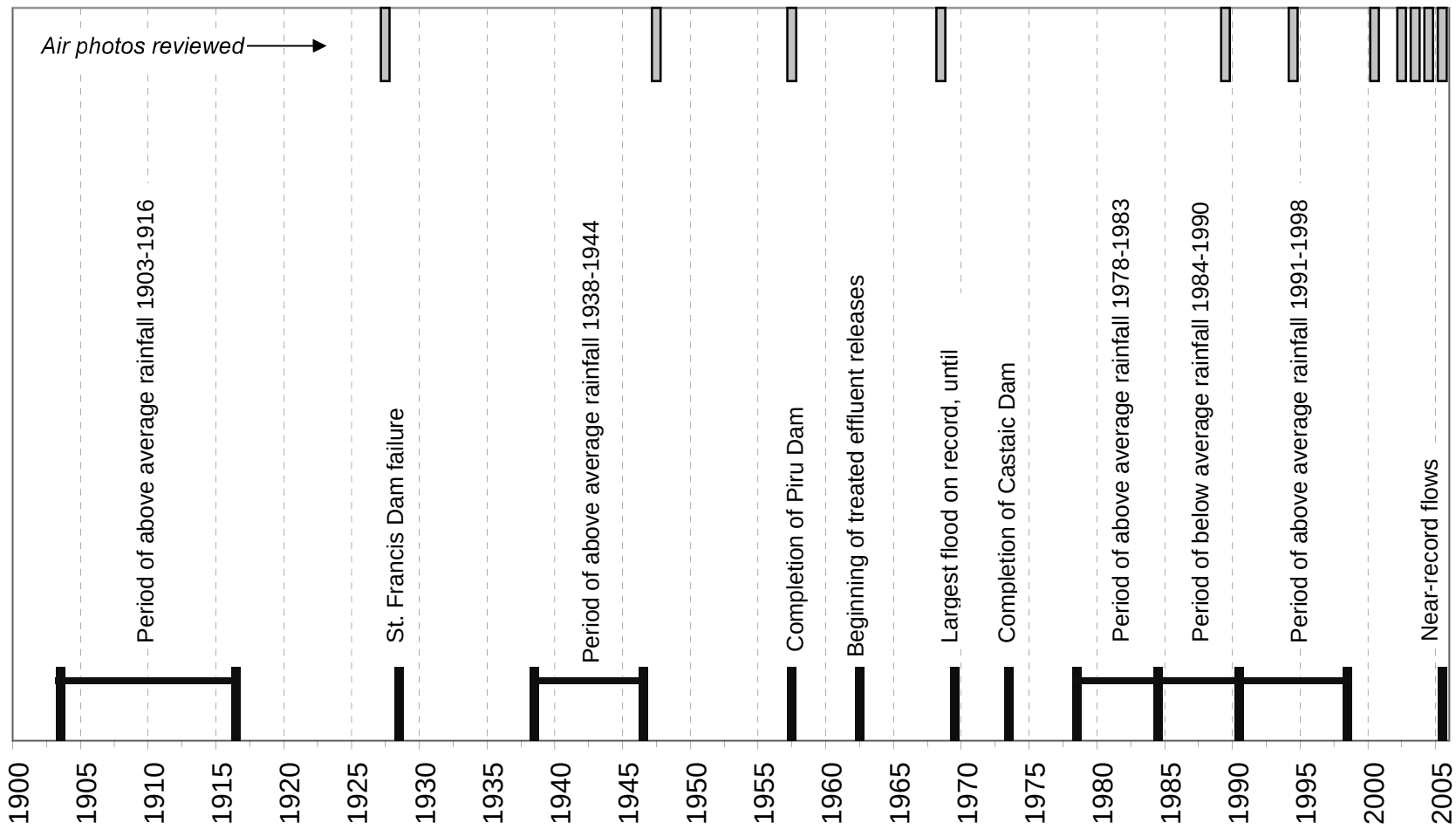

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Figure 1. Timeline of selected major events in the upper Santa Clara River, California. Also shown (at top) are the years for which aerial photographs were analyzed.

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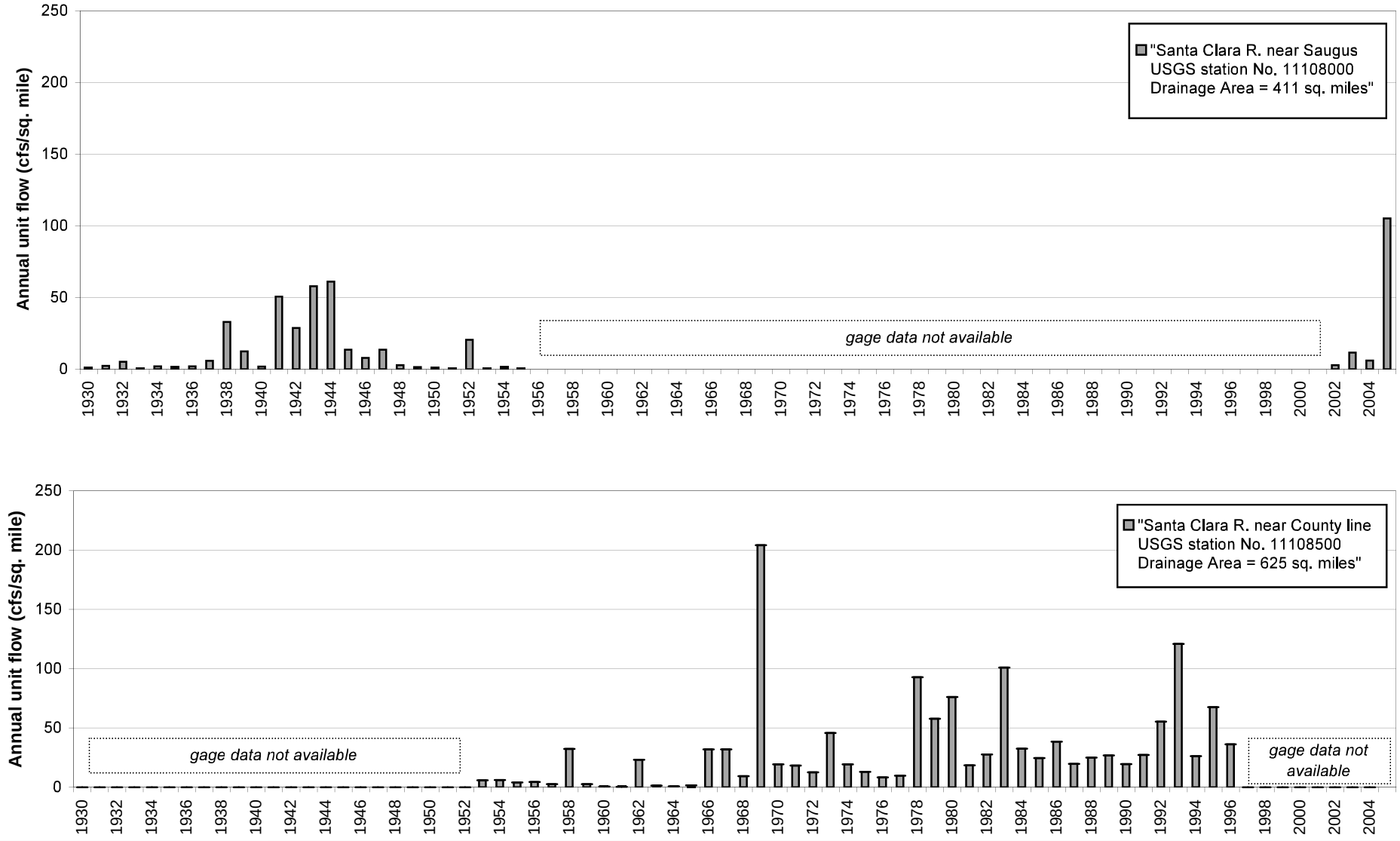

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Figure 2. Annual unit runoff (annual flow per square mile) for the Santa Clara River near Newhall at two separate gaging stations. Note that flow in drier years has increased since the 1960s, most likely due to release of treated effluent to the River.

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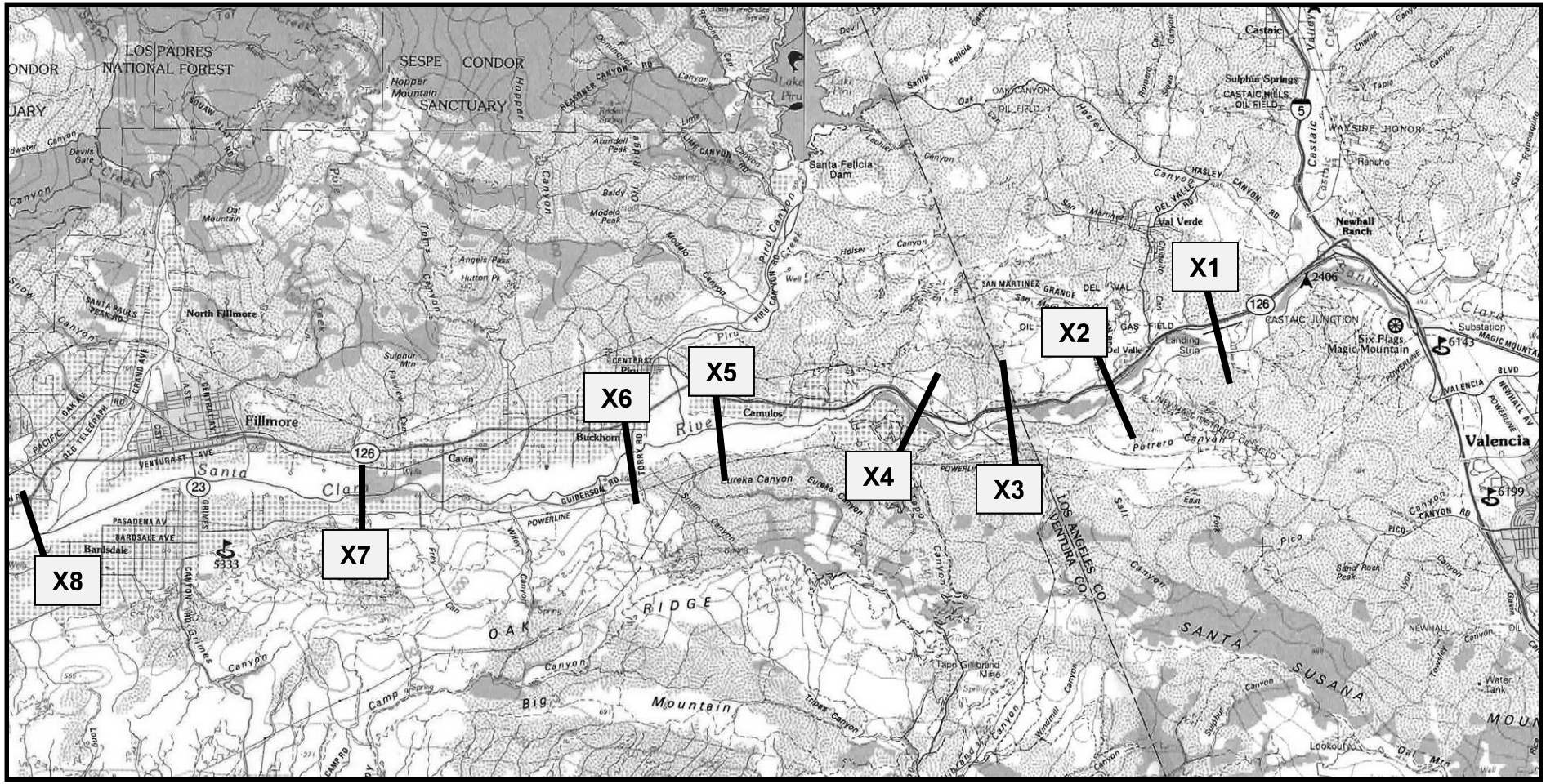

Figure 3. Location of channel cross sections on the Santa Clara River, measured on aerial photographs.

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Figure 4. Comparison of 2004 and 2005 conditions on the Santa Clara River, just downstream of the L.A./Ventura County line. Note that significant channel widening occurred in response to the 2005 events, even in heavily vegetated areas. See appendix A for photo sources.

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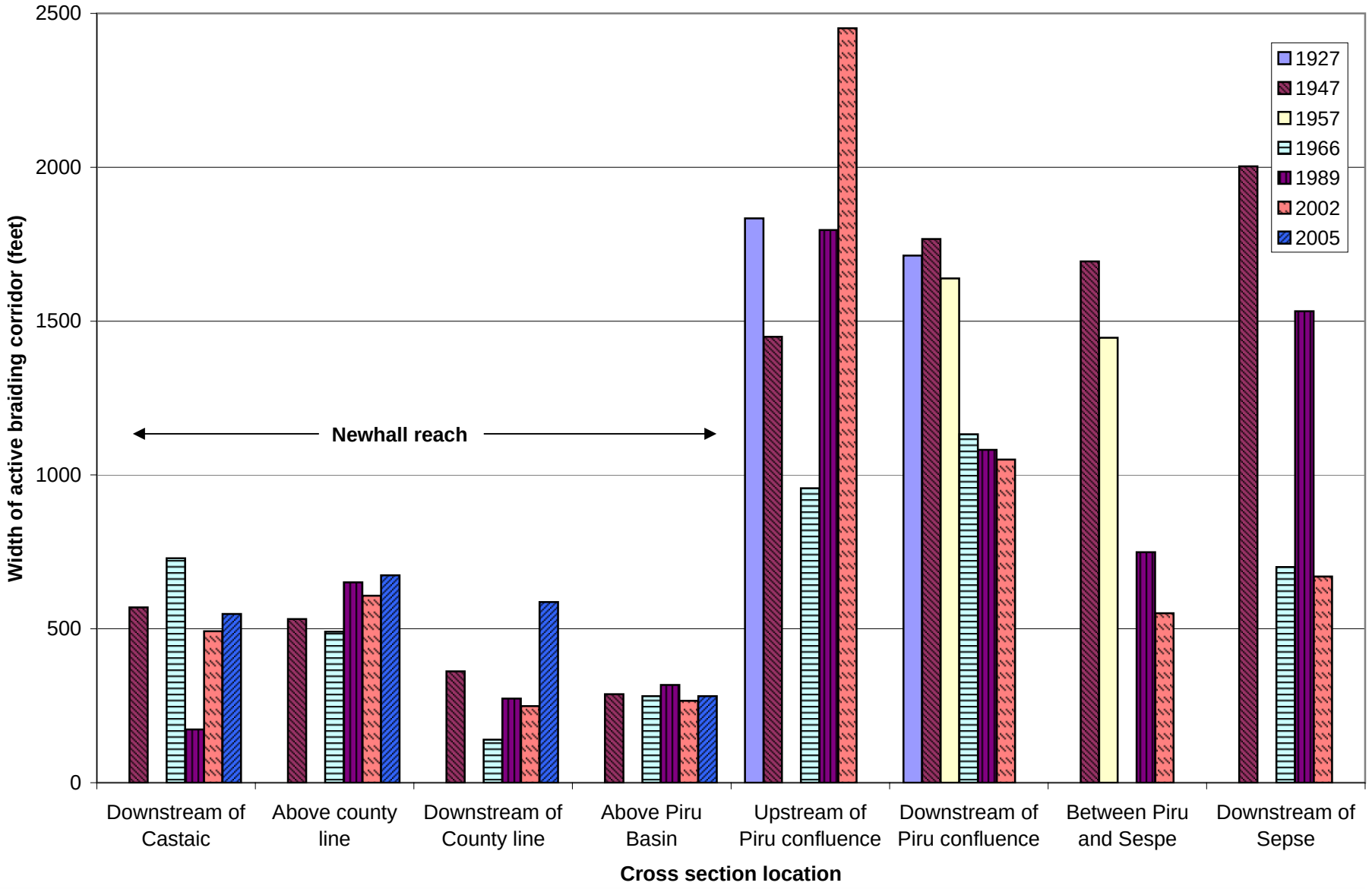

Figure 5. Measurements of active braiding corridor width from aerial photographs, for cross sections on the Santa Clara River.

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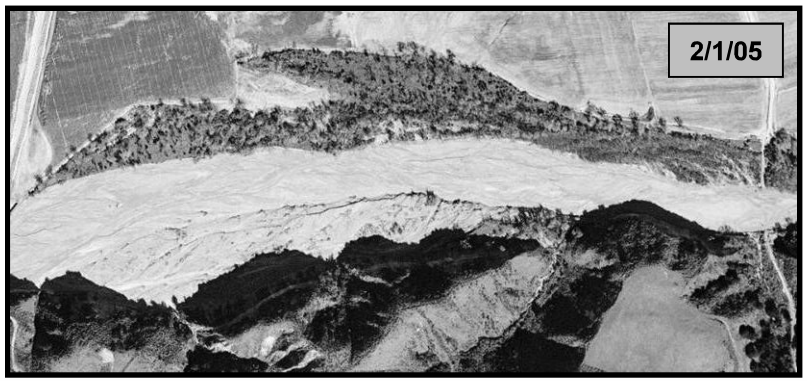

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Figure 6. Progression of aerial photographs downstream of Castaic Canyon, showing channel change between 1993 and 2005. Note that there was little change between 2000 and 2004, but the active corridor widened significantly in response to the 2005 events, and that channel traces within the active corridor were effectively erased. See appendix A for photo sources.

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APPENDICES

Appendix A: Summary of aerial photographs used for assessment of potential hydromodification effects on the Santa Clara River, Newhall, California.

Date	Number of photos	Nominal Scale	Hard Copy?	Electronic copy?	Image Type	Source/Vendor	Remarks
1927	6	2000	yes	yes	b/w	Whittier College: 80, 82, 84, F27, F28, F31	Only available photography prior to the March 1928 collapse of the Saint Francis Dam. Photos show area near Piru confluence
August 16, 1947	34	24000	no	yes	b/w - Vert Cart	USGS_GS-EM, Rolls 3, 5, 7	Previews downloaded already are sufficient.
1957	2	2000	yes	yes	b/w	Whittier College: 109, 123	1957 photos are for justdownstream of Piru Creek. Piru Dam was closed in 1957.
March 6, 1963	1 (2)	21670	no	yes	b/w - Vert Recon	USGS_ARMC630001L0049 a,b	high resolution scans
July 20, 1966	2 (4)	21670	no	yes	b/w - Vert Recon	USGS_ARM6625001L1362 a,b USGS_ARM6625001R1357 a,b	high resolution scans
August 19, 1966	1 (2)	21670	no	yes	b/w - Vert Recon	USGS_ARM6628502L1314 a,b	high resolution scans
September 13, 1966	1 (2)	21670	no	yes	b/w - Vert Recon	USGS_ARM6631405R1165 a,b	high resolution scans
November 10, 1966	2 (4)	21670	no	yes	b/w - Vert Recon	USGS_ARM6638605L1238 a,b USGS_ARM6638605L1242 a,b	high resolution scans
August 13, 1967	1	30000	no	yes	b/w - Vert Cart	USGS_AR1VBUK00010110	Preview already obtained. Downstream of Sespe Creek
May 26, 1989	5	31680	yes	yes	b/w	WAC-89CA, 27-42	LA County
						WAC-89CA, 27-62	LA County
						WAC-89CA, 27-84	LA County
						WAC-89CA, 27-109	LA County
						WAC-89CA, 27-135	LA County
May 1, 1989	6	2000	yes	yes	Color	PAS_89 06-20 PW VEN 7-229	Ventura County
						PAS_89 06-20 PW VEN 7-231	Ventura County
						PAS_89 06-20 PW VEN 7-233	Ventura County
						PAS_89 06-20 PW VEN 7-235	Ventura County
						PAS_89 06-20 PW VEN 7-269	Ventura County
						PAS_89 06-20 PW VEN 7-237	Ventura County
June 1, 1994	n/a	unknown			b/w, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
April 1, 2000	n/a	unknown	no	yes	color, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
February 1, 2002	4	Unknown	no	yes	Color, georeferenced	AirPhotoUSA (from GeoSyntec)	Covers all of Newhall project area

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Date	Number of photos	Nominal Scale	Hard Copy?	Electronic copy?	Image Type	Source/Vendor	Remarks
July 23, 2002	n/a	unknown	no	yes	Color, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
March 1, 2003	n/a	unknown	no	yes	Color, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
April 1, 2004	n/a	unknown	no	yes	Color, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
October 13, 2004	n/a	unknown	no	yes	Color, georeferenced	GlobeXplorer	downloaded select sections from LA and Ventura County
February 1, 2005	n/a	unknown	no	yes	Color, georeferenced	GlobeXplorer	only avaialable for LA County

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Appendix G

LID Water Quality Modeling Methodology

Addendum to *Appendix B* of the *Landmark Village Water Quality Technical Report* (February 2008)

INTRODUCTION

The purpose of this appendix is to describe the changes to the water quality modeling methodology that have been made to quantify low impact development (LID) BMP implementation for the Landmark Village Project (Project) (i.e., the LID Performance Standard). Changes described in this appendix are discussed in comparison to the modeling methodology that is described in Appendix B of the Landmark Village Water Quality Technical Report (Landmark Village DEIR Appendix 4.3). This appendix addresses only the elements of the modeling methodology that have been updated, added, or clarified for the quantification of LID implementation for the Project.

This appendix is organized as follows:

- Section 1 provides an overview of the changes to the modeling methodology. This section also provides clarification of the rationales for the type of model employed for this analysis.
- Section 2 describes the updates made to model input parameters, as well as the updated approaches used to develop these input parameters.
- Section 3 describes the updates to the structure of the Monte Carlo model (i.e., the way the model is set up) to account for both on-parcel BMPs and sub-regional bioinfiltration/ biofiltration facilities. This section also provides an expanded discussion of the reliability of input parameters and assumptions.

1. MODEL OVERVIEW

1.1. Overview of Changes to Model Methodology

The overall modeling methodology has not changed substantively in comparison to the methodology described in Appendix B of the Landmark Village Water Quality Technical Report (WQTR). However, the structure of the model used to represent the Project (i.e., the way the model is set up) and some model inputs have changed to represent the LID Implementation Plan. Primary changes to the model structure and inputs include:

- Parcel-based BMPs were included in the model to account for volume reduction and treatment provided in parcel-based LID BMPs prior to draining to sub-regional bioinfiltration/ biofiltration facilities.

- The representations of sub-regional bioinfiltration facilities (previously called "Project Biobasins") and the sub-regional biofiltration facility (previously called the "Project Extended Detention Basin") were updated to reflect facility designs that include retention and biofiltration components which promote infiltration.

- BMP performance statistics were updated to support the simulation of the types of BMPs in the updated BMP plan using the latest version of the International BMP Database.

- The hydrology and hydraulic modeling approach used to develop hydrologic and hydraulic inputs to the Monte Carlo model (i.e., percent capture, percent volume reduction by storm event) was enhanced to more directly derive these estimates.

In addition, the model was updated to reflect the revised Project description and associated Project land use areas. The incorporation of these changes is described in further depth in Sections 2 and 3.

1.2. Technical Basis for Modeling Methodology

While the modeling methodology has not changed substantively, this section clarifies the technical basis and provides the rationale for the continued use of this methodology to evaluate Project stormwater quality impacts.

An empirical, pollutant loads model approach has been used to assess stormwater quality impacts associated with the proposed Project. This modeling approach was selected to meet the technical requirements of the water quality impact analysis based on an extensive review of available models and a review of the available datasets applicable to the Project.

A variety of modeling approaches are capable of meeting the technical requirements of this analysis. In general, models can be grouped into three categories:

- Stochastic (or probabilistic): this type of model utilizes observed statistical patterns to produce model estimates. This type of model generally relies on empirical observations, but does not necessarily ignore causal relationships.

- Deterministic (or mechanistic, physically-based): this type of model attempts to perfectly represent physical processes and mechanisms using closed form equations derived from physical phenomena. It is noted that because these models attempt to describe systems that are inherently complex and poorly defined, most deterministic models must rely in part on empirical observations to represent causal relationships.

- Hybrid: this type of model combines elements of stochastic and deterministic models to provide more reliable model estimates.

The modeling methodology used for the Project incorporates stochastic and empirical elements, and is therefore most accurately described as a hybrid approach. The approach uses an empirical, stochastic water quality estimation approach (Monte Carlo) to produce water quality and pollutant loading estimates. Inputs to this model are derived from empirical sources (Los Angeles County Land Use Monitoring Program and the ASCE International BMP Database) and deterministic modeling of hydrology and hydraulics (EPA SWMM4.4h). This approach makes use of robust land use and BMP monitoring datasets applicable to the project and incorporates important causal relationships in hydrologic and hydraulic response that can be reliably represented with deterministic methods. This approach is believed to be most appropriate to meet the technical requirements of the impact analysis for the Project-level analysis at the tract map scale.

The literature studies summarized below generally support the use of an empirically-based hybrid approach for the type of analysis required for the Project:

- Obropta et al. (2007) evaluated six deterministic models, three stochastic models, and three hybrid approaches. They concluded that *hybrid approaches show strong potential for reducing stormwater quality model prediction error and uncertainty* [improving the ability to assess] *best management practice design, land use change impact assessment* [and other applications].

- Charbeneau and Barrett (1998) evaluated different approaches for estimating stormwater pollutant loads based on a comparison of model results to observed land use monitoring data. They found that (1) the development of accurate physically-based models *remains a difficult and elusive goal*, and current understanding of processes *is not sufficient to accurately predict event loads*, (2) a simple empirical stochastic approach is generally as reliable or more reliable than more complicated mechanistic approaches, (3) the use of land use event mean concentrations (EMCs) is appropriate for planning purposes, (4) the land use EMC approach is most reliable when land use EMCs are used as a stochastic input parameter generated from a probabilistic distribution, and (5) stormwater volume is the single most important variable in predicting pollutant loads.

- The National Research Council's (NRC) 2008 report on *Urban Stormwater Management in the United States* generally supports these findings regarding the appropriate use of stormwater quality and quantity models.

Figure 1 provides an overview of the modeling methodology, with minor updates to support the updated BMP plan shown **in bold text**.

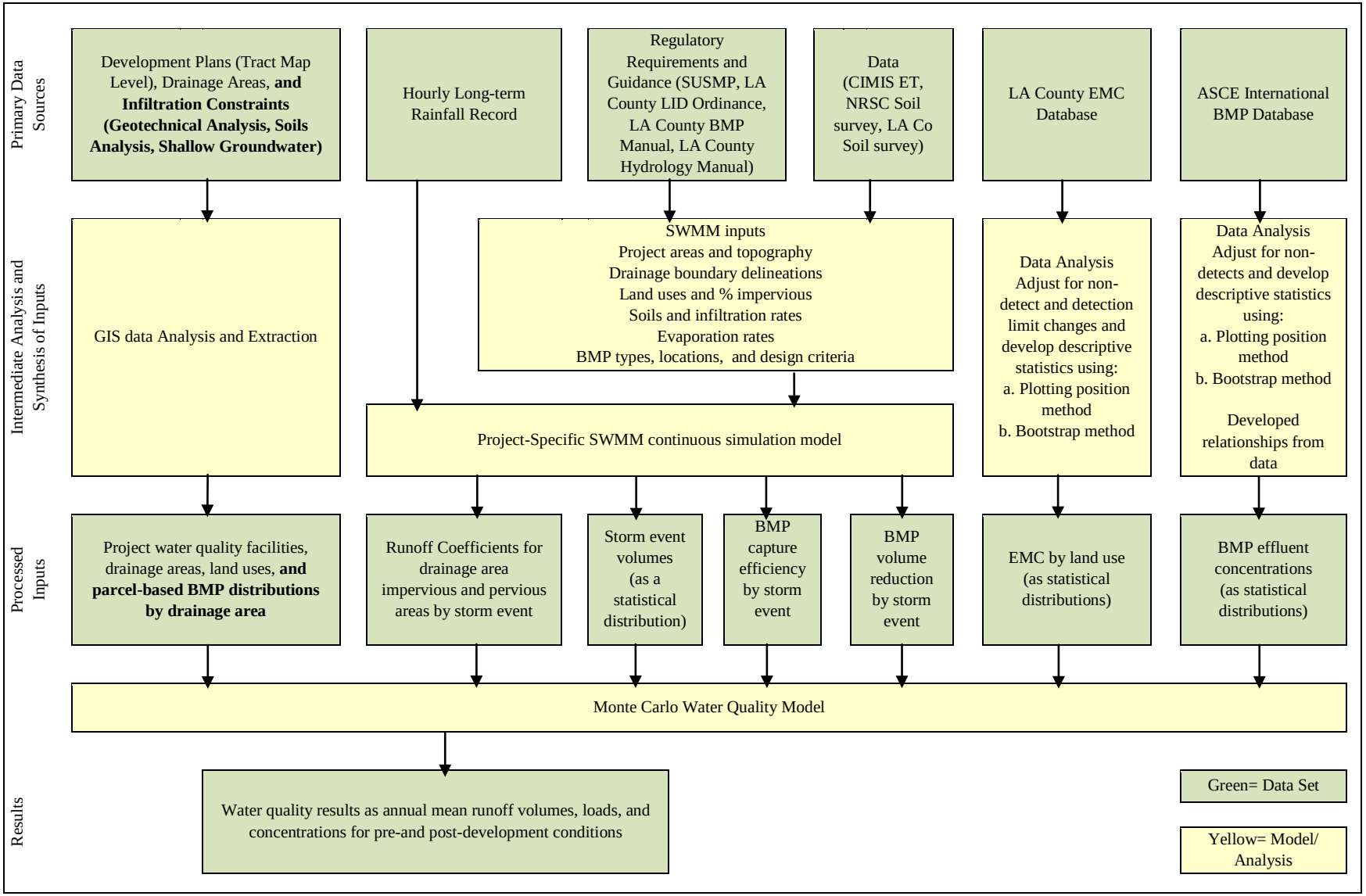

Figure 1: Overview of Updated Water Quality Analysis Methodology (Updates in Bold)

2. UPDATES TO MODEL INPUT PARAMETERS

2.1. Runoff Coefficients

As described in Appendix B of the Landmark Village WQTR, the Monte Carlo model uses runoff coefficients (derived from SWMM simulations of average drainage area soils conditions) as inputs to the modeling framework. Runoff coefficients for pervious portions of the Project area are based in part on the distribution of mapped soil properties in these areas. As a result of the change in Project development footprint, the distribution of soil properties changed slightly and triggered a re-analysis to develop runoff coefficients for pervious area. The updated soil distributions are presented in Table 1 below.

Table 1: SWMM Runoff Block Modeled Soils Distribution by WQ Drainage Area

	Sub-regional Bioinfiltration Facility		**Sub-regional Biofiltration Facility[1]**		**On-site Treatment (Media Filter or Equivalent)**		**Off-site Swales**		**Off-site Swales**	
Soil Group	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**	**Acres**	**% Total**
HSG A	43.3	21.3%	2.0	3.5%	0.8	10%	0	0%	0	0
HSG B	160.5	78.7%	54.7	96.5%	7.2	90%	8.2	100%	103.6	100%
Total	*203.8*		*56.7*		*8.0*		*8.2*		*103.6*	

[1] Includes 2.4 acres of off-site bridge.

The derivation of soil parameter assumptions for each hydrologic soil group (HSG) is summarized in Table 2. Suction head and initial moisture deficit (IMD) were estimated based on the soil texture class reported in the most Natural Resources Conservation Service (NRCS) Soil Survey (No. 675) with guidance on SWMM input parameter selection provided by James and James (2000). The saturated hydraulic conductivity (K_{sat}) was estimated based on evaluation of the ranges of undisturbed K_{sat} reported by the NRCS Soil Survey, the HSG reported by the NRCS Soil Survey with guidance on SWMM input parameter selection provided by James and James (2000), and the Los Angeles County Soil Type with infiltration characteristics described in the Los Angeles County Hydrology Manual (LA County, 2006). The preponderance of these soils data generally indicates that the project has relatively high infiltration capacity. Therefore, the assumed K_{sat} was based on the high end of the range of recommended SWMM inputs for A and B soils from James and James (2000).

Table 2: Green-Ampt Soil Parameters

Hydrologic Soil Group	Prevalent Soil Texture Class	Suction Head[1] (in)	IMD[1] (in/in)	Pre-Development K_{sat} (in/hr)	Post-Development K_{sat} (in/hr)
A	Loam	8	0.30	0.45	0.34
B	Loam	8	0.30	0.30	0.23

[1] Estimated based on texture class from Rawls, et al., (1983)

Based the soil distributions summarized in Table 1 and soil parameter assumptions summarized in Table 2, average pervious runoff coefficients were generated for each modeled drainage area to each type of sub-regional bioinfiltration/biofiltration BMP. Runoff coefficients are presented in Table 3 below. Runoff coefficients based on guidance from the Los Angeles County Hydrology Manual are also included in this table for reference (LACDPW, 2006).

Table 3: Runoff Coefficients by Water Quality Drainage Areas

WQ Drainage Area	Impervious Runoff Coefficient		Undeveloped Pervious Runoff Coefficient		Developed/Disturbed Pervious Runoff Coefficient	
	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)	Model Methodology (used for WQ model)	LA County Hydrology Manual (for comparison purposes)
Sub-regional Bioinfiltration Facilities	96.9	90	3	10	6	10
Sub-regional Biofiltration Facility	96.9	90	3	10	6	10
On-site Treatment (Media filters or equivalent)	96.9	90	3	10	6	10
On-site Biofilter Swales	96.9	90	3	10	6	10
Off-site Biofilter Swales (or equivalent)	96.9	90	3	10	6	10

2.2. Revisions to Project Land Use

Project land uses were determined from the Landmark Village Vesting Tentative Tract Map (VTTM #53108) and GIS analysis of this map (Psomas, April 2010) for the developed Project conditions, which have been revised for the Final Landmark Village EIR. In general, the assumptions regarding land use properties did not change, however, the revised land use plan contains greater information about the types of roadways than was previously available, which

required additional assumptions about the characteristic imperviousness and runoff quality from these areas, summarized below:

- For the purpose of analysis, roads adjacent to residential land uses were considered to be an integral part of single family detached land uses and assume all properties of this land use.

- Minor roads (private drives and access road) were grouped with major roads due to their proportionately small land area.

- Major roads were modeled using imperviousness and runoff quality associated with roads.

Developed conditions of the Project and associated off-site areas are summarized in Table 4.

Existing condition land uses have not changed since the previous model. Existing condition land uses can be found in Appendix B of Landmark Village WQTR.

Table 4: Developed Conditions Project and Off-site Land Uses

Land Use	Development Area (acres)		Impervious Fraction Modeled
	Project Site	Off-Site Impacts	
Modeled			
Commercial	27.3	8.0[2]	0.91
Multi-Family	82.9		0.74
Open Space	32.9		0.01
Park	10.1		0.10
Recreation	5.8		0.50
Road[1]	41.6	98.0[3]	0.91
School	9.7		0.82
Single-Family	53.9		0.42
Water Quality Facility	10.1		1.0
Not Modeled			
Open Space	18.3		NA
Total	*292.6*	*106.0*	

[1] Residential roads are included in the single-family land use. The Roads land use includes major and minor roads. Minor roads are modeled assuming the composite imperviousness and EMC of their adjacent land use types.
[2] Off-site water tanks are modeled as a commercial land use.
[3] Off-site roads consist of 2.4 acres of off-site bridge to the south and 95.6 acres of SR-126 right of way to the north. Impervious fraction of SR-126 ROW based on delineation of tentative improvement plans; not land use-based fraction.

2.3. Distribution of Parcel-based LID BMPs for Multi-Family, Commercial, Institutional, Recreation, and Park Land Uses

The LID BMP performance standard for the Landmark Village Project includes parcel-based LID BMPs in multi-family, commercial, institutional, recreation, and park land uses. There are three categories of parcel-based BMPs (Category 1, 2, and 3), the application of which depends

on infiltration feasibility constraints associated with each land use parcel. Infiltration feasibility was screened as follows to determine the distribution of Category 1, 2, and 3 parcel-based BMPs:

- Infiltration feasibility constraints were evaluated by Seward (2010) to determine areas where infiltration is likely feasible. This analysis yielded a shapefile of infiltration feasibility constraints which displays locations where infiltration is likely feasible (Category 1) and where infiltration is likely partially feasible (Category 2). No areas on the Landmark project were identified where infiltration is not feasible in any level or would be hazardous (Category 3). Criteria associated with these distinctions are described in the *Landmark Village LID Supplement.*

- The land use program described in the Landmark Village VTTM (Psomas, April 2010) was converted to a GIS shapefile and was geospatially overlain with the shapefile of infiltration constraints to determine the location(s) and area of each type of parcel-based treatment within each drainage area.

The resulting distribution of parcel-based BMPs is shown in Table 5.

2.4. Distribution of Single Family Residential Hydrologic Source Controls

The LID BMP Implementation Plan includes hydrologic source controls (HSCs) in single family detached (SFD) land uses. For the purpose of modeling, it was assumed that rooftops, patios, and walkways would be routed to pervious areas capable of managing the runoff from at least a 0.75 inch storm event.

An analysis of typical development plans was conducted to determine the portion of the impervious area in the SFD residential land use that is made up of rooftops, patios, and walkways. Based on the project VTTM land use break-down, an area-weighted lot-size of 5,025 sq-ft plus 2,250 sq-ft of residential roadway was evaluated. Based on this analysis, it was found that on average approximately 26 percent of SFD land use area is anticipated to be made up of roofs, patios and walkways.

2.5. Design of Sub-regional Bioinfiltration/Biofiltration Facilities

The LID BMP Plan includes sub-regional water quality facilities, which are proposed to manage runoff from portions of the Project that are not addressed by parcel-based BMPs as well as bypass and treated discharge from parcel-based BMPs and SFD HSCs.

The areas draining directly to sub-regional facilities and the total tributary area to sub-regional facilities are provided in Table 5 below. Sub-regional bioinfiltration facilities are distributed around the Project site and have individual drainage areas. Because all sub-regional bioinfiltration facilities will be designed to the same design and performance standards, all area draining to sub-regional bioinfiltration facilities was considered to be part of on WQ drainage area for the purpose of modeling.

Table 5: Areas Draining to Parcel-based BMP Types within WQ Drainage Areas

WQ Drainage Area	Category 1		Category 2		SFD HSC[1]		Directly to WQ Facility		*Total*	
	Area (Ac)	Imp (%)	Area (Ac)	Imp (%)	Area (Ac)	Imp (%)	Area (Ac)	Imp (%)	Area (Ac)	Imp (%)
Sub-regional Bioinfiltration Facilities	36.1	77.7	50.0	73.4	28.1	50.0	89.6	41.1	203.8	56.8
Sub-regional Biofiltration Facility	2.8	78.0	36.3	81.9	0	0	17.6	71.3	56.7	78.4
On-Site Treatment (Media Filters or Equivalent)	No parcel-based BMPs						8.0	44.9	8.0	44.9
On-site Biofilter Swales (or equivalent)	No parcel-based BMPs						8.2	91.0	8.2	91.0
Off-site Biofilter Swales (or equivalent)	No parcel-based BMPs						103.6	91	103.6	91.0

1 - Includes single-family roofs, patios and sidewalks, draining to a pervious area with an equivalent square-footage.

2.6. Updated BMP Performance Parameters

As in the previous modeling methodology, the performance of project BMPs is estimated as a function of three factors: (1) the fraction of stormwater runoff receiving treatment (often referred to as percent of runoff captured, or simply percent capture); (2) the pollutant removal achieved in the BMP by virtue of infiltration and/or evapotranspiration (generically referred to as volume reduction); and (3) the pollutant removal achieved in the BMP by virtue of improved water quality. The performance parameters associated with these factors have been updated to reflect the LID BMP Plan as described in the sections below.

2.7. BMP Capture Efficiency and Volume Reduction

The Monte Carlo model utilizes event-by-event estimates of BMP capture efficiencies and volume reduction to describe the hydrologic and hydraulic performance of BMPs. These inputs were developed using SWMM simulations. While this approach has not fundamentally changed, slight changes were required to accommodate the LID BMP Plan including (a) the simulation of parcel-based BMPs that are "nested" within the drainage area of sub-regional facilities, (b) the simulation of SFD HSCs that provide volume reduction from SFD land uses, and (c) the simulation of sub-regional bioinfiltration/ biofiltration facilities that combine infiltration and biofiltration elements in different proportions depending on infiltration feasibility in the location of the facility. The approaches for developing capture efficiency and volume reduction inputs for the Monte Carlo model for each of these BMP types are described in the sections below. These approaches make use of the SWMM Runoff block (hydrologic simulation module) and the SWMM Storage/Treatment block (hydraulic simulation module), both operated in continuous simulation mode for a period of 40 years.

2.7.1. **Parcel-based BMPs**

Estimates of capture efficiency and volume reductions achieved by parcel-based BMPs were developed based on hydraulic representations of parcel-based BMPs in EPA SWMM4.4h (Storage/Treatment block), with spatially-averaged tributary catchments (Runoff block). A hypothetical spatially-averaged catchment representation was used because exact drainage areas and imperviousness for each parcel-based BMP are not available at this level of analysis (i.e., Tier 2, the tract map project scale). The hypothetical spatially-averaged catchment was assigned an area of one acre and an impervious fraction representative of the composite imperviousness of Project areas draining to parcel-based BMPs. This catchment was simulated in the SWMM Runoff block to produce a characteristic runoff hydrograph, which was routed through each type of parcel-based BMPs using the SWMM Storage/Treatment block. The reliability of the spatially-averaged catchment approach is discussed in Section 3.2.

The hydraulic representation of each type of parcel-based BMP was developed in the SWMM Storage/Treatment block based on a standard BMP profile formulated to result in the maximum feasible infiltration of the 0.75 inch design storm for each infiltration constraint condition. The standard profiles are primarily dependent on the design infiltration rate of underlying soil for each of the constraints categories. Based on an assessment of likely infiltration rates and allowable infiltration volumes (Seward, 2010), the design infiltration rates were selected as follows.

- **Category 1** areas are located in areas identified as having a natural, undisturbed infiltration rate of greater than 0.5 inches per hour and the potential to use direct infiltration or dry wells to infiltration. Direct infiltration was assumed to be feasible in areas with depth of fill less than 10 feet. Dry wells were assumed to be feasible in areas where the depth from the bottom of fill to seasonally-high groundwater is greater 10 feet. The design infiltration rate for this category of parcel-based BMP was selected by applying a reduction factor of 25 percent to the low end of estimated infiltration rate. The result is a design infiltration rate of **0.375 inches per hour**.

- **Category 2** areas are generally located in areas with natural, undisturbed infiltration rate of less than 0.5 inches per hour (Seward, 2010) and/or where depth of fill or separation from the bottom of fill to groundwater would not permit full infiltration of the design storm volume. The design infiltration rate for this category of parcel-based BMP was selected by applying a reduction factor of 50 percent to the low end of estimated infiltration rate. The result is a design infiltration rate of **0.25 inches per hour**. This assumption considers physical limitations of infiltration into compacted and low permeability soils as well as hazards associated with introduction of excess water into fill structures.

Based on these design infiltration rates and the design goals for parcel-based BMPs described above, the geometric inputs to the SWMM hydraulic representations of parcel-based BMP are described in the Table 6 below.

Table 6: SWMM Hydraulic Representation of Parcel-based BMPs

Parameter	Units	Parcel-based BMP Categories	
		Category 1	Category 2
Surface Ponding Depth below Overflow	ft	1.0	0.5
Media Depth	ft	1.5	1.5
Design K_{sat} of Amended Media	in/hr	2.0	2.0
Design K_{sat} of Underlying Soil	in/hr	0.375	0.25
Thickness of Gravel Layer	ft	0	1.5
Height of Underdrain Invert Elevation above Bottom of BMP	ft	None	1.5
Depth of Retention Storage[1]	inches	18.3	9.0
BMP Footprint as Fraction of Impervious Area	ac/ac	3.1%	2.9%
Average Annual Capture Efficiency (Percent Capture)	-	53%	80%
Average Annual Volume Reduction of Captured Water (Percent Volume Reduction)	-	100%	41%
Average Annual Reduction in Runoff Volume	-	53%	33%

[1] Retention storage depth is determined based on the equivalent depths of volume retained in ponding, media, and gravel (i.e. the full storage volume of Category 1 and, for Category 2, volume below underdrain), as well as additional retention storage in media.

The storm-by-storm capture efficiency and volume reduction estimated from the parcel-based BMP simulations was extracted from SWMM model output and used to represent the hydraulic performance of these BMPs in the Monte Carlo model.

2.7.2. **SFD Hydrologic Source Controls**

The effect of HSCs was simulated by routing runoff from impervious areas to pervious areas within the SWMM Runoff block (hydrologic simulation module) and tabulating the combined runoff coefficient from this area for each storm event. For the purpose of analysis, it was assumed that impervious areas would be routed over an equal amount of pervious area with properties modified to represent amended soils in the areas receiving runoff. Table 7 provides the model parameters that were used to represent SFD HSCs.

Table 7: SWMM Model Representation of Hydrologic Source Controls

Parameter	Units	Assumption	Basis of Assumption
Impervious to Pervious Ratio	ft	1:1	Based on typical available landscape area per tributary area, or equivalent HSC
Slope of Pervious Area	ft/ft	0.05	See Appendix B
Depression Storage of Pervious Area	inches	0.5	Based on soil amendments to 4 inch depth improving soil moisture storage capacity by 0.125 inches per inch; actual design of HSCs may vary
Manning's Surface Roughness of Pervious Area	-	0.25	See Appendix B (tables)
K_{sat} of Pervious Area	in/hr	Based on drainage area average developed K_{sat}	See Appendix B (tables)
Saturated Hydraulic Conductivity of Pervious Area (K_{sat})	in/hr	Varies based on soil type	See Table 2
Suction Head of Pervious Area	inches	8.0	See Table 2
Initial Moisture Deficit of Pervious Area	in/in	0.3	See Table 2
Runoff coefficient of impervious plus pervious area	-	13.3	Modeled in SWMM. Takes into account infiltration of runoff from impervious area in pervious area.

The effect of HSCs was accounted in the Monte Carlo model by modifying the runoff coefficient of the areas being disconnected and receiving disconnection. The runoff coefficient of this area was tabulated from SWMM output for each storm event.

2.7.3. **Sub-regional Bioinfiltration/Biofiltration Facilities**

The hydraulic performance of each sub-regional bioinfiltration/biofiltration facility is dependent on characteristics of the tributary drainage area (including the amount of parcel-based BMPs and HSCs provided in the tributary drainage area), the volume of the facility, the underlying design infiltration rate, and the outlet control configuration. Therefore, to evaluate the capture efficiency and volume reduction performance of sub-regional facilities, drainage area hydrologic representations and facility hydraulic representations were developed for each facility.

Drainage Area Representation for Sub-regional Facilities

The drainage area representation used to evaluate sub-regional facilities was developed using the same approach described in Landmark Village WQTR Appendix B, with the exception that the effects of "nested" parcel-based BMPs and SFD HSCs were approximated by embedding hydrologic elements in the drainage area representation to represent these BMPs (i.e., hydrologic BMP representations).

To approximately account for the effects of parcel-based BMPs in each sub-regional facility drainage area, "hydrologic representations" of parcel-based BMPs were used. These representations do not account for detailed hydraulic routing, but generally account for the effect

of parcel-based BMPs on the overall volumetric response from the drainage area. These representations included increasing the depression storage of selected pervious and impervious areas, and routing impervious area runoff to these "sump" areas based on the distribution of Category 1 and 2 LID BMPs in each WQ drainage area described in Table 5.

To ensure that this representation provides a reasonably accurate approximation of the effects of parcel-based BMPs, the volume reductions resulting from this hydrologic representation were compared to the volume reductions resulting from the more detailed hydraulic representations described in Section 2.1.7.1. The pervious or impervious depression storage values used in the hydrologic representations were adjusted such that the average annual volume reductions due to depression storage losses (i.e., hydrologic representations of parcel-based BMPs) were equivalent to the average annual volume reductions achieved in the hydraulic representations of parcel-based BMP. The adjusted impervious or pervious depression storage depths used for the drainage area hydrologic representations of parcel-based BMP are reported in Table 8 below. The reliability of this approach is discussed in Section 3.2.

Table 8: SWMM Hydrologic Model Representation of Parcel-based BMPs

SWMM Runoff Parameters	Units	Parcel-based LID BMP Type	
		Category 1	Category 2
Depression storage, pervious	inches	21	10
Depression storage, impervious	inches	NA	NA
Imperviousness	%	0	0
Infiltration Rate	in/hr	0.375	0.15
Average Annual Reduction in Runoff Volume from Hydrologic Representation	-	53%	33%
Average Annual Reduction in Runoff Volume from Hydraulic Representation (See Table 6)	-	53%	33%

The selected footprint areas of the parcel-based BMPs for these hydrologic representations were determined by scaling the footprint areas generated from the hydraulic parcel-based BMP representations based on the impervious fraction of the drainage area.

To represent the hydrologic effects of SFD HSCs in the sub-regional facility drainage area representation, the portions of the drainage area attributed to SFD rooftops, patios, and walkways was routed over pervious landscape areas in a one-to-one ratio. Parameters used to represent this disconnection scenario are reported in Table 7.

Hydraulic Representation of Sub-regional Facilities

Sub-regional bioinfiltration/biofiltration facilities were represented in the SWMM Storage/Treatment block based on the proposed designs of these facilities.

Designs were developed by first estimating the infiltration rate of soils underlying the proposed facility location and identifying any other constraints on infiltration (Table 9).

Table 9: Sub-regional Facility Design Infiltration Rates

Facility Type	Assumed Design Infiltration Rate, inches per hour	Basis for Assumption
Sub-regional Bioinfiltration Facilities	0.25	Seward screening shows areas have infiltration rates greater than 0.5 in/hr. Factor of safety of 2 applied.
Sub-regional Biofiltration Facility	0.25	Seward screening shows areas have infiltration rates greater than 0.5 in/hr. Factor of safety of 2 applied.
On-site Treatment (Media filter of equivalent)	No infiltration assumed	Are may be treated by a variety of proprietary systems that do not promote infiltration.
On-site and Off-site Biofilter Swales	NA - Infiltration not modeled explicitly	Because designs of biofilter swales have not been developed for all areas, estimates of volume reductions expected in swales were derived from analysis of the International BMP Database.

A standard profile for each sub-regional facility was then developed based on the portion of the facility volume that can be dedicated to infiltration and the portion of the facility volume that is treated and released. This is a function of the design infiltration rate of soil under the facility.

Finally, the geometry of the basins was determined via iterative model runs to meet the following criteria:

- Surface storage draws down in less than or equal to 48 hours (subsurface storage in the pore spaces of gravel and suction storage in media pores may persist for longer than 48 hours as this storage does not pose a risk related to vector control or habitat creation).

- The facility captures and retains or treats runoff volumes such that less than 20 percent of the baseline drainage area runoff volume "bypasses" the facility (i.e., is routed around the facility or flows through the facility without significant treatment). The baseline drainage area runoff volume is defined as the volume that would occur without parcel-based BMPs or SFD HSCs. Limiting the sub-regional facility bypass to 20 percent of the baseline volume ensures that the Project performance standard of 80 percent capture is achieved on a drainage area basis, including the volume reduction effect of BMPs that are nested in the drainage area plus the volume reduction and treatment provided in the downstream sub-regional facility.

After an iterative solution was found that meets these criteria, the capture efficiency and volume reduction were tabulated for each storm event by post-processing SWMM model output. The estimated capture efficiency and volume reduction on a storm-by-storm basis were used to describe hydraulic performance of sub-regional facilities in the Monte Carlo model.

Sub-regional infiltration/ biofiltration facility type and geometries are listed in Table 10.

Table 10: Sub-regional Bioinfiltration/Biofiltration Facility Geometry

Parameter	Units	Sub-regional Bioinfiltration Facilities	Sub-regional Biofiltration Facility
Facility Type	--	Shallow vegetated basins with vegetated media filtration and gravel sump below underdrain	Basin-type BMP with extended detention, biofiltration, and incidental infiltration
Facility Volume	ac-ft	Varies by facility	4.3
Surface Ponding Depth below Overflow	ft	1.5	6
Surface Drawdown Time	hours	9	48
Assumed Design Saturated Hydraulic Conductivity (K_{sat}) of Underlying Soil	in/hr	0.25	0.25
Assumed Biofiltration Media Thickness	ft	2 or greater	1.5 or greater
Water Equivalent Retention Depth below Underdrain	ft	0.5	None
Subsurface Drawdown Time	hours	24	NA

Table 12 reports long-term hydrologic performance of sub-regional facilities (capture efficiency and volume reduction) as well as the overall drainage area capture and volume reduction inclusive of volume reductions achieved in nested parcel-based BMPs, hydrologic source controls, and sub-regional facilities.

Table 11: Sub-regional Facility Hydraulic Performance and Drainage Area Total Performance

WQ Drainage Area	Total Tributary Area	Composite % Imp	Sub-Regional Facility Capture Efficiency of Runoff Volume	Sub-Regional Facility Volume Reduction of Captured Water	Parcel-based Volume Reduction Upstream of Sub-regional Facility	Drainage Area Total Capture Efficiency	Drainage Area Total Volume Reduction
Sub-regional Bioinfiltration Facilities	203.8	57%	71%	33%	32%	80%	48%
Sub-regional Biofiltration Facility[1]	56.7	78%	74%	16%	24%	80%	34%
On-site Treatment (Media filter or equivalent)	8.0	45%	80%	0%	0%	80%	0%
Biofilter Swales (on-site)	8.2	91%	80%	20%	0%	80%	20%
Biofilter Swales (off-site)	103.6	91%	80%	20%	0%	80%	20%

[1] Includes 2.4 acres of off-site roadway (bridge) that drains to this BMP.

2.8. BMP Pollutant Removal

The Monte Carlo model characterizes BMP pollutant removal as a function of BMP effluent quality (statistical distributions and irreducible concentration) derived from analysis of the International BMP Database (www.bmpdatabase.org). To support the updated BMP plan, the latest version of the BMP Database (obtained 1/13/2011) was queried and analyzed to produce effluent quality distributions characteristic of the types of BMPs included in the updated BMP plan. Project BMP types were matched to the most representative category of BMP in the BMP Database for the purpose of modeling (Table 12).

Table 12: BMP Effluent Quality Performance Parameters

BMP Type	Facility Type	BMP has Treated Effluent?	BMP Database Category for Effluent Quality
Sub-regional Bioinfiltration Facilities	Infiltration and Biofiltration	Y	Media Filter
Sub-regional Biofiltration Facility	Extended detention, biofiltration and incidental infiltration	Y	Media filter plus detention basin treatment train
Media Filters or equivalent	Flow-based BMPs incorporating media filtration or equivalent treatment mechanisms	Y	Media Filter
Biofilter Swales (on- and off-site)	Biofilter Swales	Y	Biofilter
Parcel-based Category 1 LID BMPs	Infiltration	N	NA
Parcel-based Category 2 LID BMPs	Infiltration and Biofiltration	Y	Media Filter
SFD HSC	Infiltration and ET	N	NA

NA – BMP does not have treated effluent.

Table 13 summarizes the number of data points (individual storm events) and percent non-detects for the pollutants and BMP types of interest for which sufficient data were available. Table 14 summarizes the log-normal statistics that were used in the water quality model, and Table 15 summarizes arithmetic descriptive statistics for those data sets. Table 16 summarizes the irreducible effluent concentration estimates used by for water quality modeling of the proposed condition. A full description of the statistical analysis methods and assumptions used to generate BMP descriptive statistics is contained in Appendix B of the Landmark Village WQTR Appendix B (February 2008). Note that because of a paucity of data in the BMP Database for some pollutants, no treatment was assumed for nitrite (NO_2), total aluminum, and chloride, so these constituents are not included on the following summary charts even though they were included in the model. Load reductions are still possible for these pollutants via volume reduction provided in BMPs.

Table 13: Summary of Number of Data Points and Percent Non-Detects for BMP Effluent Concentration Data from the International BMP Database

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
Media Filter	Count	294	292	135	99	263	186	254	185
	% ND	9%	8%	39%	3%	3%	7%	30%	21%
Detention Basin	Count	509	258	85	93	178	173	197	174
	% ND	0%	3%	7%	12%	4%	32%	46%	9%
Biofilter (Swale)	Count	461	547	361	312	499	255	455	255
	% ND	2%	2%	15%	0%	0%	3%	35%	10%

Table 14: International BMP Database Lognormal Statistics of BMP Effluent Concentrations

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
Media Filter	Mean	2.54	-2.30	-2.59	-1.10	-0.24	1.46	0.89	3.07
	St. Dev	1.24	1.06	1.27	1.15	1.01	1.15	1.26	1.12
Detention Basin	Mean	2.56	-1.75	-2.25	-2.09	-0.08	1.36	1.88	2.81
	St. Dev	1.25	1.23	0.92	1.66	1.39	0.95	1.19	1.03
Media Filters plus Detention Basin Treatment Train[1]	Mean	2.54	-2.30	-2.59	-2.09	-0.24	1.46	0.89	3.07
	St. Dev	1.24	1.06	1.27	1.66	1.01	1.15	1.26	1.12
Biofilter (Swale)	Mean	2.66	-1.78	-2.44	-1.68	-0.30	1.81	0.91	2.76
	St. Dev	1.06	1.17	1.37	1.01	0.94	0.69	1.36	1.05

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

Table 15: International BMP Database Arithmetic Estimates of BMP Effluent Concentrations

BMP		TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
	units	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*ug/L*	*ug/L*	*ug/L*
Media Filter	Mean	27.54	0.18	0.17	0.65	1.32	8.28	5.42	40.25
	St. Dev	52.86	0.26	0.34	1.08	1.77	13.65	10.75	63.56
Detention Basin	Mean	28.29	0.37	0.16	0.49	2.43	6.10	13.30	28.12
	St. Dev	54.70	0.69	0.19	1.90	5.95	7.41	23.40	38.68
Media Filters plus Detention Basin Treatment Train[1]	Mean	27.54	0.18	0.17	0.49	1.32	8.28	5.42	40.25
	St. Dev	52.86	0.26	0.34	1.90	1.77	13.65	10.75	63.56
Biofilter (Swale)	Mean	25.24	0.34	0.22	0.31	1.16	7.81	6.26	27.44
	St. Dev	36.53	0.58	0.52	0.41	1.39	6.14	14.39	38.72

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

Table 16: International BMP Database Arithmetic Irreducible Effluent Concentration Estimates

BMP	TSS	TP	NH3	NO3	TKN	Diss Cu	Tot Pb	Diss Zn
	mg/L	*mg/L*	*mg/L*	*mg/L*	*mg/L*	*ug/L*	*ug/L*	*ug/L*
Media Filter	1.49	0.03	0.01	0.01	0.24	0.99	0.20	1.35
Detention Basin	2.99	0.09	0.34	0.02	3.78	15.16	0.27	78.83
Media Filters plus Detention Basin Treatment Train[1]	1.49	0.03	0.01	0.02	0.24	0.99	0.20	1.35
Biofilter (Swale)	2.02	0.07	0.01	0.66	0.19	2.03	0.32	4.95

1- MF/DB treatment train is based on lower of observed effluent quality between these two BMP categories. Treatment train effluent quality based on MF for all constituents except nitrate-nitrogen, which is based on DB.

3. MONTE CARLO MODEL

3.1. Updates to Model Methodology

The Monte Carlo model framework used to simulated the LID BMP Implementation Plan is identical to that described in Appendix B of the Landmark Village WQTR, however, the model structure (i.e., the way the model is set up) has been modified somewhat to account for the volume and pollutant load reductions achieved through “nested” parcel-based BMPs and SFD HSCs upstream of sub-regional facilities. Accounting for these nested BMP requires another “loop” of pollutant load generation, removal and routing algorithms to be implemented in the model within each sub-regional facility drainage area for each simulated event. An overview of the revised model structure to account for “nested” BMPs is illustrated in Figure 2.

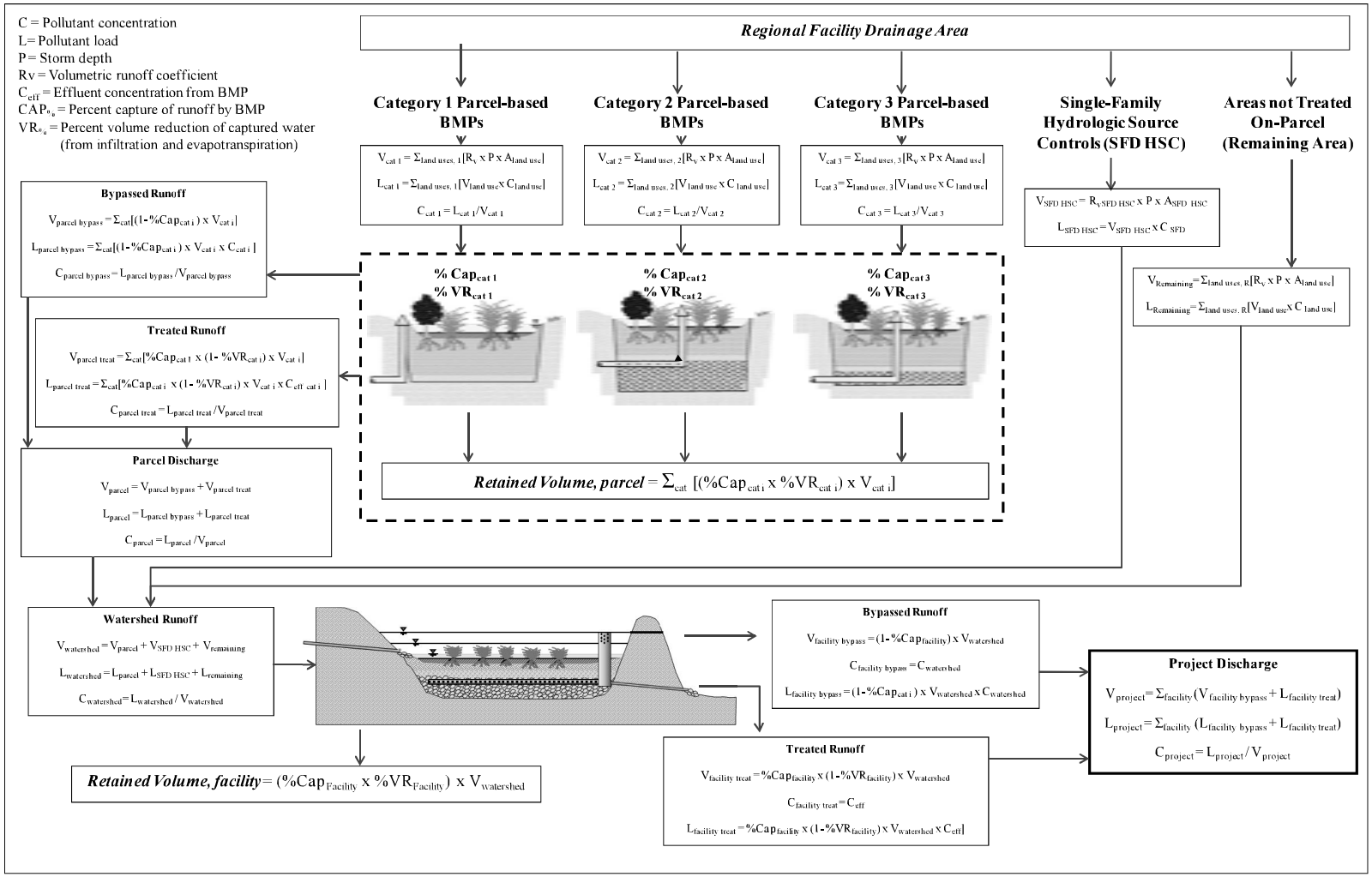

Figure 2: Monte Carlo Model Schematic with Nested LID BMPs and Hydrologic Source Controls (Generalized Schematic)

3.2. Model Parameter Reliability and Assumptions

This section discusses the reliability of new or revised model parameters and assumptions necessary to support the LID BMP Plan.

3.2.1. Drainage Area Runoff Coefficients and Hydrologic Parameter Sensitivity

The estimation of runoff coefficients is highly dependent on soil properties (i.e., infiltration potential) and less dependent on parameters such as evapotranspiration (ET) rates, slopes, and surface roughness. Soil properties are estimated as accurately as possible from available soils data, incorporating the latest soil survey conducted by the USDA NRCS as well as locally-developed infiltration relationships provided in the Los Angeles County Hydrology Manual (LACDPW, 2006). The resultant estimates of runoff coefficients that may somewhat overestimate or underestimate stormwater runoff.

Table 17 provides a comparison of assumed project runoff coefficients (developed from SWMM modeling) to applicable references.

Table 17: Comparison of project runoff coefficients to applicable references

Drainage Area Imperviousness	**Project Runoff Coefficient Assumptions** (varies by drainage area; range provided)	**LA County Hydrology Manual** (Minimum Cu = 0.1)	**Ventura County Manual,** Silty Clay Soils (Soil Types 2 or 3)	**ASCE/WEF Manual of Practice 23/87** (3rd order polynomial)
Reference	Table 3	LACDPW, 2006	Ventura County, 2010	ASCE/WEF, 1998
90% impervious, developed	0.88	0.82	0.87	0.73
60% impervious, developed	0.6 - 0.62	0.58	0.61	0.41
30% impervious, developed	0.33 - 0.36	0.34	0.36	0.23
1% impervious, undeveloped	0.03 - 0.07	0.11	0.11	0.05

Based on the comparison provided in Table 17, the assumed developed condition runoff coefficients are reasonably consistent with applicable references, although generally somewhat high. Assumed undeveloped condition runoff coefficients are generally lower than applicable references. The combined effect of these trends results in a somewhat higher estimate of impacts associated with the Project and somewhat higher estimate of absolute runoff volumes and associated pollutant loads in the proposed condition. As such, the assumed runoff coefficients are believed to be somewhat conservatively selected and reliable for the purpose of impact analysis.

3.2.2. **Parcel-based BMP Infiltration Feasibility Screening**

The types of parcel-based BMPs applied to commercial, multi-family, institutional, recreation and park land uses was determined based on infiltration feasibility constraints, as described in Section 2.1.3. The criteria used to categorize parcel-based treatment based on infiltration constraints are in agreement with the infiltration constraints listed in the *Ventura Technical Guidance Manual* (Ventura County, 2010), the *LA County LID Ordinance and Manual* (LACDPW, 2009), and the *LID BMP Design, Investigation and Reporting Requirements Administrative Manual* (LACDPW, 2011). Constraints were mapped as accurately as possible at the Tier 2 level of analysis. More detailed site investigation performed at later project phases may result in somewhat different distributions of parcel-based BMPs.

3.2.3. **Parcel-based BMP Infiltration Rates**

Infiltration rates beneath parcel-based BMP were assumed based on input from project geotechnical consultant (Seward, 2011) based on review of geologic information and proposed sources of fill material. While it is expected that infiltration rates may vary across the Project, the assumed values are believed to be representative of anticipated average conditions. Detailed designs will be supported by site-specific infiltration testing and will generally be based on the same design goals used to develop the parcel-based BMP designs simulated in this analysis.

3.2.4. **Parcel-based BMP Model Representations**

For the purpose of estimating the characteristic hydraulic performance (capture efficiencies and volume reductions) of parcel-based BMPs, detailed hydraulic representations were simulated to manage runoff from hypothetical spatially-averaged catchments. The spatially-averaged hypothetical approach provides representative and reliable estimates of hydraulic performance for two key reasons. First, the sizes of parcel-based BMPs scale linearly with tributary impervious area, there it is expected that the nearly identical capture efficiency and volume reduction (as a percent of total runoff volume) would be expected for catchments with a wide range of tributary area impervious fraction. Second, parcel-based BMP designs include significant “equalization storage” above their treatment layer, therefore the effect of catchment size (i.e., time of concentration) is not believed to be sensitive in the estimation of hydraulic performance. Therefore the use of a hypothetical, spatially-averaged catchment is appropriate to generate these inputs.

In order to size parcel-based BMPs for the purpose of analysis, BMP geometries were assumed based on the assumed underlying infiltration rate and the Project design goals for parcel-based BMPs. While the geometry assumed for this analysis is specific to a certain BMP design, the resulting performance parameters derived from this representation are reasonably representative of the hydraulic performance of a wide range of parcel-based BMPs provided that the Project design goals for parcel-based BMPs remain the same.

3.2.5. **Drainage Area Routing and Nested BMP Representations**

Each sub-regional facility drainage area includes areas treated by parcel-based BMPs and/or SFD HSCs. Because the exact location and detailed designs of these parcel-based BMPs and HSCs are not known at the Tier 2 level of analysis, it would be inappropriate to simulate detailed drainage area hydraulic routing to account for these nested BMPs. However, it would also be inappropriate to ignore the role of nested BMPs in the hydraulic performance of downstream region infiltration/ biofiltration facilities. The approach described in Section 2.1.7.3 balances these considerations to provide a reliable estimate of the hydraulic performance of sub-regional facilities that is consistent with the Project performance standards.

3.2.6. **Sub-regional Facility Infiltration Rates and Model Representations**

Infiltration rates in the locations of proposed sub-regional bioinfiltration/biofiltration facilities were estimated based on geologic information, soils data, and limited infiltration testing results available at the time of analysis. To account for uncertainty in these estimates, substantial factors of safety were applied. As such, it is believed that infiltration rates are somewhat conservatively selected for the purpose of this analysis and it is anticipated that higher design infiltration rates may be supported through site-specific analysis conducted at the time of the final hydrology report. Should detailed testing show infiltration rates are lower than assumed, additional design features such as dry wells and/or selectively graded fill material could be used to achieve at least the assumed design infiltration rate.

3.2.7. **BMP Effluent Statistics**

BMP effluent concentrations are based on studies contained in the most recent version of the International BMP Database. These studies are screened to remove data for undersized (i.e., inadequate design criteria) BMPs that are likely to have pollutant removal performance substantially less than the BMPs to be constructed for the Project. This screening is believed to improve the accuracy of BMP performance estimates; however, it is only intended to remove BMPs that are clearly unrepresentative in terms of sizing. The screening process is intended to include BMPs with adequate performance that may not be as well designed or maintained as the structural BMPs for the Project. It is anticipated that the BMPs for the Project will perform as well, if not better than, the projected performance based on the ASCE International BMP Database.

3.2.8. **Assumption of No Correlation between Model Parameters**

The water quality model randomly selects stormwater pollutant concentrations independent of the storm depth or antecedent dry period for each storm event modeled. The validity of the assumption of independence between variables is supported in Appendix B of the Landmark Village WQTR Appendix B. In general, no consistent level of correlation has been demonstrated between stormwater EMCs and rainfall depth or the antecedent dry period.

The assumption of independence of model parameters is believed to result in representative or somewhat conservative estimates post-developed runoff quality and loading, as well as

somewhat conservative estimates of Project impacts. First, the empirical distribution of runoff EMCs implicitly includes events with a wide range of antecedent dry periods and event sizes. Therefore, the effects of antecedent dry period and storm depth are implicitly reflected in model estimates. Second, where weak correlations have been observed, concentrations tend to decrease with increasing storm depth. Because bypass from BMPs tends to occur more frequently in larger events and at the end of events, the assumption of no dependence would generally result in higher bypass concentrations, on average, than would be expected if these negative correlations were included. On these bases, random selection of stormwater pollutant concentrations, independent of storm depth and antecedent dry period, is believed to be the most reliable option for the modeling methodology at this level of analysis.

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