VOLUME I OF FIVE VOLUMES
TEXT, TABLES AND FIGURES

GEOTECHNICAL INVESTIGATION UPPER LLAGAS CREEK FLOOD PROTECTION PROJECT SANTA CLARA COUNTY, CALIFORNIA

Prepared for
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3614

July 27, 2006

URS

55 South Market Street, Suite 1500
San Jose, California 95113

Project Number 28649692

URS

July 27, 2006
Project Number 28649692

Mr. Bal Ganjoo
Senior Project Manager
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

Subject: Geotechnical Engineering Report
Upper Llagas Creek Flood Protection Project (ULFPP)
Morgan Hill, California
Agreement Number A2951A, File Number 2898

Dear Mr. Ganjoo:

As authorized, we have prepared the accompanying Geotechnical Engineering Report for the ULFPP geotechnical investigation. This report was prepared in accordance with Subtask "3.2 Geotechnical Report" of Agreement Number A2951A dated August 25, 2005. Pacific Geotechnical Engineering, as a subcontractor to URS, performed engineering services during field exploration and prepared the geologic and seismic portions of the report.

The ULFPP is located in southern Santa Clara County, California, encompassing portions of the communities of Morgan Hill, San Martin, and portions of unincorporated Santa Clara County. Llagas Creek is one of the tributaries of the Pajaro River and drains a 104 square-mile watershed within Santa Clara County. The ULFPP will provide improved flood protection along approximately 14 miles of East Little Llagas Creek, West Little Llagas Creek, and Llagas Creek.

The U.S. Army Corps of Engineers (Corps), as the lead federal agency, is responsible for planning, design, and construction of the channel improvements. The Santa Clara Valley Water District (District), as the local sponsor, is responsible for land rights acquisition, utility and structure relocations, and design and construction of the box culverts and bridge.

We have submitted draft reports dated May 22, 2006, June 19, 2006 and July 7, 2006. The July 7, 2006 report includes revisions based on the District's and Corps' review comments transmitted to us by e-mail on June 2, 2006, and during a telephone conference call on June 27, 2006. This final report includes revisions transmitted to us by telephone and e-mail on July 17 and July 18, 2006, respectively.

In general, we have made revisions consistent with your June 2, June 27, and July 18, 2006 review comments. However, there are a few comments we wish to clarify, listed as follows (using the same comment numbers as the District's).

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- Comments 42, 51 and 74. Comment 42: "General-No discussion of Erosion Potential of Channels????" Comment 51: "The draft geotechnical report does not present any recommendations for erosion control or slope protection measure. I believe, especially the cut slope should be provided with appropriate slope protection measure, such as rock riprap, geotextile fabric, etc to prevent toe erosion/scour during high velocity water flow." Comment 74: "The overall stability of the levees could also be dependent on their susceptibility to surface erosion due to excessive current velocities. Past reports and photographic records of a flood control project downstream of the site indicate that surface erosion was a major contributor to the levee damage and its subsequent failure. One explanation for this erosion was that the vegetation had been largely removed exposing the levees and overbanks and making them susceptible to erosion. Analysis should be done to address this. Should there be any reason to believe that the stability of the levees at the project site could be affected by surface erosion."

  Response: Erosion potential of channels and slope protection measures are beyond the scope of work in the Agreement. This work item was deleted during negotiations of the Agreement. Erosion potential is a key component that would be more appropriately addressed when the hydraulic model is refined in subsequent stages of the design.

- Comment 46. "The results of all slope stability analysis have (been) presented in a single table under the title, "Levee Slope Stability Evaluation." Because 17 cross sections analyzed are not all levee sections, it would be preferable to present the results in two separate tables, one for levee section, and the other for cut slope."

  Response: We have revised the left hand column of the tables in Section 4 to include Levees 6A and 6B. Therefore, the new levees and berms are identified in the table. We would prefer not to separate each table into two tables, i.e. one for levees and one for cut slopes; otherwise this would require rewriting Section 4.

- Comment 67: "For ease of reading and illustrative purposes, please summarize and show the results of pseudo-static analysis by having plot(s) of factor of safety vs. seismic coefficient."

  Response: Our approach to evaluate the earthquake case was based on a 3-step evaluation: (1) pseudo-static screening based on a seismic coefficient of 0.15g, (2) pseudo-static analysis based on a seismic coefficient = ½ PGA if Step 1 factor of safety is less than 1.0, and (3) determine the permanent deformation of slope using the critical slip surface in Step 2. Since all the Step 1 screening factors of safety are greater than 1.0, plotting the factors of safety against seismic coefficient will yield a straight vertical line in a factor of safety versus seismic coefficient plot. Therefore, it was mutually agreed that such a plot is unnecessary.

- Comment 68: "The use of 'drained strength' parameters in cohesive soils assumes that no positive excess pore pressure develops during failure (Hamilton Wetlands Restoration Project, New Hamilton Partnership Levee report, URS San Francisco Office, 2004). Assuming the above comment is applicable to the project site conditions, explain how these drained parameters were to be used in light of the fact that even after construction an undrained condition may still be present. It has been shown that the drained analysis in cohesive soils could be estimated following SHANSEP principles (Ladd and Foott, 1974) by using undrained strength parameters."

  Response: It is our opinion that the drained strength parameters we selected for the ULFPP site are appropriate. Along the ULFPP alignment, the clays encountered are stiff to hard. For stiff clays, the long-term stability of slopes is governed by the fully softened shear strength (approximated by high strain triaxial tests) defined by the effective ϕ' and the steady porewater pressure condition (after "Soil Mechanics in Engineering Practice," 3rd Edition, Terzaghi, Peck and Mesri, 1996). Therefore, it is not in conflict with Ladd's comments on the New Hamilton Partnership Levee Report.

- Comment 77. "Because the basis of the geotechnical design and recommendations are based on preliminary design drawings and hydraulic analysis (Corps), please comment on the effects of stability factor of safety on increased/decreased levee build height and channel embankment slope. Perhaps include some form of chart/curve that illustrates the build height and reflectance on factor of safety/settlement. The intent of this comment is to provide flexibility should the civil design plans change."

  Response: Beyond the authorized scope of work in the Agreement.

The accompanying report presents our opinions and recommendations regarding the geotechnical aspects influencing design and construction of the proposed improvements. The opinions and recommendations have been based upon our review of available information, the results of a field investigation, laboratory testing, engineering analyses we conducted as part of our scope, as well as engineering judgment and local experience.

Subsequent to our draft report dated July 7, 2006, the Corps requested that their preliminary design plans be included in this report. For convenience of reference, these plans are included in Appendix J (U.S. Army Corps of Engineers Preliminary Design Plans) of Volume V.

Mr. Pitipat Preedonant, P.E., Senior Staff Engineer, assisted in the selection of the engineering parameters, the engineering analysis of levee stability, potential levee seepage, and potential settlement, as well as preparation of the report. Dr. G. Reid Fisher, C.E.G., of Pacific Geotechnical Engineering prepared the discussion on geology and seismicity including fault locations and distances; Mr. Mark Schmoll, C.E.G. and Mr. Ivan Wong, Principal Seismologist of URS provided peer review of this subject. Mr. Paul Boddie, G.E., Principal-In-Charge, provided peer review for all geotechnical aspects of the project, except for earthquake engineering; Mr. Robert Green, G.E., provided peer review of the latter. This peer review was in accordance with the "Final Project Work Plan and Quality Control Procedures, Geotechnical Investigation, Upper Llagas Creek Flood Protection Project" dated November 22, 2005 by URS.

We would like to give special recognition to Mr. Daniel J. Peluso, G.E., of Pacific Geotechnical Engineering. Mr. Peluso served as the primary coordinator of all field exploration/field activities and conducted the initial review of samples recovered in the field. In addition, Mr. Peluso compiled existing geotechnical data from Pacific Geotechnical Engineering's files. His efforts expedited the field exploration program and provided valuable insight regarding local subsurface conditions.

We are pleased to be of service to you on this project. If any questions should arise, please contact our office.

Sincerely,

S. Stephen Huang

Stephen Huang, G.E. 02150
Deputy Project Manager

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cc: Pacific Geotechnical Engineering; Mr. Daniel J. Peluso

TABLES

FIGURES

APPENDICES

SECTION 1

1.1 GENERAL

This report presents the results of a geotechnical investigation for the Upper Llagas Creek Flood Protection Project in Santa Clara County, California. Included are (1) a review of the available subsurface information and literature, (2) the subsurface exploration and laboratory testing program conducted for this study, and (3) the results of engineering evaluations that serve as the basis for our opinions and recommendations regarding the geotechnical engineering aspects of the project. The report is organized in five volumes, as described below.

- Volume I of Five Volumes – Text, Tables and Figures
- Volume II of Five Volumes – Appendices A (Previous Investigations), B (Exploratory Borings) and C (Piezometers)
- Volume III of Five Volumes – Appendices D (Laboratory Testing Program) and E (Laboratory Testing Program of Remolded Levee Fill)
- Volume IV of Five Volumes – Appendices F (Slope Stability), G (Settlement Calculations), H (Steady State Seepage Analyses) and I (Liquefaction Analyses)
- Volume V of Five Volumes – Appendix J (U.S. Army Corps of Engineers Preliminary Design Plans)

The locations and layout of the project are shown in Figure 1, Location Map, Figure 2, Site Map and in Figure 3, Reach Location Plan; the latter figure presents the location of the six reaches: Reach 4, Reach 5, Reach 6, Reach 7, Reach 8 and Reach 14. The Site Map, Sheet Index and Drawing List is presented in Figure 4. A more detailed layout of the project, including the approximate centerline with stationing presented in feet, is shown on the Soil Boring Site Plans, Figures 5 through 27.

The as-drilled locations of borings and piezometers were surveyed by the Santa Clara Valley Water District (District) and a summary of the horizontal coordinates and ground surface elevations is presented in Figure 28, Survey Coordinates Soil Boring Locations. Based on the survey, the District also plotted borings and piezometers at locations shown in the Soil Boring Site Plans, Figures 5 through 27.

Throughout the text of this report, numerical values will be presented in metric units; English units are also presented in parenthesis. However, most graphs, charts and laboratory test results are presented in English units. Furthermore, stationing, elevation, depths and reach lengths are presented exclusively in English units.

All coordinates shown and survey work performed are based on the California State Plane Coordinate System, Zone 3 NAD 1983 and NAVD 88.

For Reach 7 there is a discrepancy of centerline stationing described as follows. The "Corps Hydraulic Stationing" is used in the Corps Sediment Assessment and Hydraulic Design Office Report including the Appendices, the Bestor survey information, and the District's Soil Boring Site Plan. However, the "Corps Design Stationing" is used in the Corps design plans (plan and profile sheets) and the Corps Design Cross Sections. Unless noted otherwise, all stationing used in this report for all six reaches is the "Corps Hydraulic Stationing".

Unless stated otherwise, left or right directions are determined looking upstream.

1.2 PROJECT DESCRIPTION AND UNDERSTANDING

1.2.1 Introduction

The U.S. Army Corps of Engineers (Corps) is currently designing the Upper Llagas Creek Flood Protection Project (ULFPP). The goal of the ULFPP, when completed is to provide an increased level of flood protection (100-year for urban areas of Morgan Hill and between five to ten years for agricultural areas of San Martin and unincorporated Santa Clara County) with adequate freeboard between Buena Vista Road and 800 feet upstream of Wright Avenue. For purposes of this Agreement, the current design plan available from the Corps will be referred to as the Preliminary Plan. The Preliminary Plan generally consists of plan and profile sheets (no water surface levels are shown), typical cross-sections with limited dimensions, and the Draft Sediment Assessment and Hydraulic Design Office Report.

The geotechnical investigation and analysis for the ULFPP entails the following:

- Perform field investigations and laboratory testing to provide design criteria and recommendations for the Preliminary Design and future Final Design.
- Perform engineering analyses and develop conclusions and recommendations for the Preliminary Design.

Upon completion of this study, in a separate Agreement, the District will use the field investigation and laboratory testing data results from the current investigation to perform additional analysis and develop conclusions/recommendations for future final design and construction of the ULFPP project.

The ULFPP is located in southern Santa Clara County, California, encompassing portions of the communities of Morgan Hill, San Martin, and portions of unincorporated Santa Clara County. Llagas Creek is one of the tributaries of the Pajaro River and drains a 104 square-mile watershed within Santa Clara County. The ULFPP will provide improved flood protection along approximately 14 miles of East Little Llagas Creek, West Little Llagas Creek, and Llagas Creek.

The Corps, as the lead federal agency, is responsible for planning, design, and construction of the channel improvements. The District, as the local sponsor, is responsible for land rights acquisition, utility and structure relocations, and design and construction of the box culverts and bridge.

The following information regarding the Preliminary Plan of the ULFPP was taken from the "Draft Sediment Assessment and Hydraulic Design Office Report, U.S. Army Engineer District, San Francisco Corps of Engineers", dated December 2002. This information summarizes details given in the report. Where there is a conflict between the information summarized and the report, the information given in the report will be used.

For engineering and construction purposes, the ULFPP is divided into six reaches: Reach 4, Reach 5, Reach 6, Reach 7, Reach 8 and Reach 14. Locations of the reaches are included in the table below.

Reach No.	Extent	Length	Creek
4	From Buena Vista Avenue to East Little Llagas	2.4 miles	Llagas Creek
5	East Little Llagas to 700-feet upstream of Highway 101	0.4 mile	Llagas Creek
6	700-feet upstream of Highway 101 to 1,000-feet upstream of West Little Llagas Creek (to end of diversion channel)	3.7 miles	Main Branch Llagas Creek
7	From Main Branch Llagas Creek to West Dunne Avenue	2.6 miles	West Little Llagas Creek
8	From West Dunne Avenue to 800-feet upstream of Wright Avenue in Morgan Hill	1.1 miles	West Little Llagas Creek
14	Main Branch Llagas Creek to 600-feet upstream of East Middle Avenue	3.4 miles	East Little Llagas Creek
	Total	**13.6 miles**	

A number of new culverts are also planned in Reach 7 and Reach 8, as summarized in the tables below.

Reach 7 Location	Proposed Design						Existing	
	Roadway Width (ft)	Culvert Size #-w (ft) x h (ft)	From Station (ft)	To Station (ft)	Length (ft)	Type of Crossing	Roadway Width (ft)	Culvert Size w (ft) x h (ft)
Spring Avenue	60	3-10x9	133+30	133+90	60	RCB*	60	
West Dunne to Ciolino Avenue (Morgan Hill Plaza)	-	1-8x8	147+70	141+00	674	RCB	-	

*RCB-Reinforced Concrete Box

Reach 8 Location	Proposed Design						Existing		
	Roadway Width (ft)	Culvert Size #-w (ft) x h (ft)	From Station (ft)	To Station (ft)	Length (ft)	Type of Crossing	Roadway Width (ft)	Culvert Size w (ft) x h (ft)	Type of Crossing
5th Street	60	2 - 10x9	152+60	152+00	60	RCB	60	5x5	RCB
4th Street / Monterey Hwy	270	2 - 10x9	157+00	154+30	270	RCB	270	9x6	RCB
3rd Street	14	2 - 10x9	160+70	161+30	60	RCB	14	14x7	RCB
2nd Street / Del Monte Avenue	250	2 - 10x9	165+00	167+50	250	RCB	250	10x5	RCB
Warren Avenue	40	2 - 10x9	170+00	170+50	50	RCB	40	10x5	RCB
Main to Wright along Hale Ave (Future Santa Teresa	2200	2 - 10x8	178+70	199+25	2,200	RCB	N/a	-	-

Reach 8 Location	Proposed Design						Existing		
	Roadway Width (ft)	Culvert Size #-w (ft) x h (ft)	From Station (ft)	To Station (ft)	Length (ft)	Type of Crossing	Roadway Width (ft)	Culvert Size w (ft) x h (ft)	Type of Crossing
Expwy)									
Main Street	N/a	-	-	-	-	-	70	9x5	RCB
Wright / Hale Avenue	N/a	-	-	-	-	-	110	60"	RCP**

**RCP-Reinforced Concrete Pipe

In addition, one 3.5m (11.5 foot) wide driveway bridge is planned at about Station 114+70 (Reach 7).

Details of each reach are summarized below.

1.2.2 Reach 4: East Little Llagas Creek to Buena Vista Avenue

Design Flow: This reach will be designed for current capacity as well as new capacity to preclude induced flooding from upstream improvements, approximately five-year flood protection.

Location: Llagas Creek between Station 501+96 (East Little Llagas Creek) and Station 377+32 (Buena Vista Avenue), approximately 2.4 miles in length.

Summary of Proposed Improvements: The proposed design for Reach 4 consists of deepening and widening the channel. The channel widening, in general, is limited to the left side of the channel (looking downstream). The proposed design consists of a trapezoidal cross section that has a 25 feet bottom width, 1V:3H side slopes and a 9 to 13 feet depth. The average channel velocity for this reach is 4.8 feet per second. Maintenance access will be from the top of bank.

1.2.3 Reach 5: 700 Feet Upstream Highway 101 to East Little Llagas Creek

Design Flow: This reach will be designed for existing capacity as well as new capacity to preclude induced flooding from upstream improvements, approximately ten-year flood protection.

Location: Llagas Creek from Station 501+96 (East Little Llagas Creek) to Station 525+50 (700 feet upstream of Highway 101), approximately 0.4-mile long.

Summary of Proposed Improvements: The proposed design for Reach 5 consists of deepening and widening the channel. The channel widening is limited to the left side of the channel (looking downstream). The design includes a restored, more natural channel with a vegetated and minimally maintained left bank (looking downstream) bench 26 feet wide, with 1V:3H side slopes, and a 2-year channel forming discharge channel with a bottom width of 30 feet and 6 to 7 feet deep. The average design channel velocity in this reach is 4.6 feet per second. Maintenance access would be from the top of bank.

1.2.4 Reach 6: Diversion Channel to 700 Feet Upstream Highway 101

Design Flow: This reach will be designed for existing capacity as well as new capacity to preclude induced flooding from upstream improvements.

Location: Reach 6 is subdivided into two reaches, Reach 6A (0.5 mile long) and 6B (3.2 miles long). Reach 6A is located between Station 718+59 (West Little Llagas Creek diversion channel) and Station 689+84 (Monterey Hwy). Reach 6B spans the length of Llagas Creek from Station 689+84 (Monterey Highway) to Station 525+50 (700 feet upstream of Highway 101).

Summary of Proposed Improvements: The proposed design for Reach 6A consists of approximately 2,560 feet of set back levee on the left overbank (looking downstream). The proposed set back levee will leave the existing channel undisturbed. The right overbank (looking downstream) is in naturally high ground and eventually ties into Silveira Lake. The average design channel velocity in this reach is 4.2 feet per second. The proposed levee will provide maintenance access to the area and consists of 1V:2H side slopes with a 12 foot top width, varying from 5 to 6 feet in height.

The proposed design for Reach 6B consists of a two-stage flood control channel, with left and/or right benches, requiring deepening, widening and filling of the existing channel. Proposed channel widening is limited to left side of the channel (looking downstream) where possible. A restored, more natural channel with a vegetated left bank (looking downstream) bench 26 feet wide, with either 1V:3H or 1V:4H side slopes and 2-year channel forming discharge channel, with a bottom width of 30 to 36 feet is proposed throughout most of Reach 6B. The average design channel velocity in this reach is approximately 4.5 feet per second. Maintenance access will be along the top of bank.

1.2.5 Reach 7: Dunne Avenue to Llagas Creek

Design Flow: This reach is designed for 100-year flood protection.

Location: Reach 7 is divided into two reaches. Reach 7A (1.3 miles long), is located from Station 78+75 (La Crosse Drive) to Station 11+00 (Main Branch Llagas Creek). Reach 7B (1.3 miles) is located between Station 147+35 (West Dunne Avenue) and Station 78+75 (La Crosse Drive). Reach 7 is approximately 2.6 miles long.

Summary of Proposed Improvements: Reach 7A is a diversion channel from West Little Llagas Creek to Llagas Creek and consists of two channel configurations: a trapezoidal section with levees and a two-stage flood control channel, with left and right benches. The diversion channel will be new earth channel construction.

- From Station 11+00 (the confluence) to Station 31+00 (the downstream face of Middle Avenue), the diversion channel will have a trapezoidal cross section with a 30-foot bottom width, 1V:2H side slopes and a 12-foot depth, with levees on both banks.
- From Station 31+00 (upstream of Middle Avenue) to Station 79+00 (the downstream face of the southern La Crosse Drive bridge), the proposed channel consists of approximately 2,700 feet of a two-stage flood control channel. The 2-year effective discharge channel has a 30-

foot bottom width, 1V:2.5H or 1V:3H side slopes and a depth of 6 feet, where it then transitions to a high flow area of the channel with 8-foot wide benches on the left and right side of the channel, 1V:2.5H side slopes and a top width of 120 feet. The average channel velocity in this section of the reach is 4.0 feet per second. Maintenance access will be along the top of bank.

Reach 7B is upstream of Reach 7A and the proposed design consists of two channel configurations: a trapezoidal cross section and a two-stage flood control channel, with left and/or right benches. The 2-year channel-forming discharge channel has a 25 to 50-foot bottom width, 1V:3H to 1V:2.5H side slopes and a depth of 4.5 to 5 feet, where it then transitions to a high flow area of the channel with 8 to 20 foot wide benches, 1V:3H side slopes and a top width of 110 to 145 feet. The proposed trapezoidal cross section would have a 25-foot wide bottom width, 1V:2.5H side slopes and a 9 to 13-foot depth. The average channel velocities in this section of the reach are 3.0 to 4.0 feet per second. Maintenance access would be along the top of bank.

The proposed design requires 5,764 feet of levee in Reach 7A and 2,606 feet of levee in Reach 7B. The levee height varies from 5 to 6 feet, with 1V:2H side slopes and a top width of 12 feet. A maintenance access road and trail will be located on top of the levees.

A fish barrier will be located near Station 14+00 in Reach 7A.

The proposed design includes two new culverts and a driveway bridge as follows:

- An 11.5-foot wide driveway bridge (at approximately Station 114+70)
- A 60-foot long, 10-foot wide by 9-foot deep box culvert at Spring Avenue and
- A 674-foot long, 8-foot wide by 8-foot deep box culvert from Ciolino Avenue to Dunne Avenue.

1.2.6 Reach 8: Downtown Morgan Hill

Design Flow: This reach is designed for 100-year flood protection.

Location: West Little Llagas Creek in downtown Morgan Hill between Station 147+35 (West Dunne Avenue) to Station 207+50 (the project limits, approximately 800 feet upstream of Wright Avenue). The Reach is approximately 1.1 miles long.

Summary of Proposed Improvements: Seven existing undersized culverts are to be replaced with new culverts 10 feet wide by 9 feet deep (except as noted) at the following locations: 5th Street, 4th Street / Monterey Hwy, 3rd Street, 2nd Street / Del Monte Avenue, Warren Avenue, Main to Wright along Hale Avenue (10 feet wide by 8 feet deep).

- Between Station 147+35 (West Dunne Avenue) and Station 178+70 (Main Avenue), the proposed design consists of approximately 3,000 feet of trapezoidal vegetated gabion channel consisting of a 20-foot bottom width, 1V:0.5H side slopes, depth of 7.5 to 10 feet, and a maximum top width of 30 feet. The average design channel velocity in this section is 5.4 feet per second.

- Between Station 178+70 (Main Avenue) and Station 199+25 (Wright Avenue), the proposed design consists of replacing the existing creek with two 10-foot wide by 7 to 8-foot high reinforced concrete box culverts under the roadway. The total proposed length will be approximately 2,200 feet.
- Between Station 199+25 (Wright Avenue) and Station 207+50 (the upstream limit of project near Hillwood Lane), the proposed design consists of approximately 600 feet of trapezoidal channel with a cross section of a 20-foot bottom width, 1V:3H side slopes, a depth of 8 feet, and a top width of 70 feet. The average design channel velocity in this section is 2.8 feet per second.

Reach 8 does not have the ability to convey the channel capacity plus freeboard in several locations. Flood walls will be necessary at these locations. Between Station 151+00 and Station 175+50 approximately 1,700 feet of flood walls 1.5 to 2.5 feet high will be built on both sides of the creek.

1.2.7 Reach 14: East Little Llagas Creek

Design Flow: This reach is designed for existing capacity as well as capacity to preclude induced flooding from upstream improvements, 10-year flood protection.

Location: East Little Llagas Creek from Station 0+00 (the confluence with Llagas Creek) to Station 180+00 (just upstream of East Middle Avenue) approximately 3.4 miles long.

Summary of Proposed Improvements: The proposed design for Reach 14 consists of deepening and widening the channel using a trapezoidal cross section. Maintenance access will be along the top of bank and will include maintaining the existing trail located on the top of the right bank, from the confluence with Llagas Creek to Sycamore Avenue.

- From Station 0+00 (the confluence with Llagas Creek) to Station 10+00 (the fish barrier), the proposed trapezoidal cross section has a 50-foot bottom width, 1V:3H side slopes and a 14-foot depth. The average design channel velocity for this section of Reach 14 is 3.3 feet per second.
- At Station 10+00, a 5-foot high vertical V-notch concrete drop structure will serve as a fish barrier.
- From Station 10+00 (the fish barrier) to Station 124+00 (Sycamore Avenue), the proposed trapezoidal channel continues, with a 50-foot bottom width, 1V:3H side slopes and 11-foot depth. The average design channel velocity for this section of Reach 14 is 5.0 feet per second.
- Upstream of Station 124+00 (Sycamore Avenue), the creek runs parallel to Highway 101. The embankment for Highway 101 also acts as the right bank of the creek (looking downstream). To avoid heavy modification to the Highway 101 embankment, it is proposed that the channel be modified. The conceptual design consists of a trapezoidal channel with a varying bottom width of 15 to 40 feet and 1V:3H side slopes that tie into the existing embankment. Because the channel was relocated in the 1970's, the existing channel was created with the use of levees on the left bank from Station 131+00 to 144+00. To avoid

drastic modification of the existing levees and the embankment, the channel bottom width will only increase from 15 to 20 feet. However, due to the higher peak flow in this area, the levees will need to continue down to Sycamore Avenue from Station 131+00 to Station 124+50. The average design channel velocity for this section of Reach 14 is 4.0 feet per second.

- From Station 172+00 (upstream of Middle Avenue) to Station 180+00 (the confluence with West Little Llagas Creek), the proposed channel configuration has a 40-foot bottom width, 1V:3H side slopes and 7-foot depth. West Little Llagas Creek enters East Little Llagas Creek via a culvert under Highway 101. The average design channel velocity for this section of Reach 14 is 4.0 feet per second.

With the proposed design, the design water surface elevation is above natural ground at several locations. A minimum of 1 foot of freeboard will be used. Levees or berms 2 to 5 feet high will be necessary to contain the 10 percent flow at these locations.

1.2.8 Llagas Creek around Lake Silveira (Junction of Reach 6 and Reach 7A)

The current design proposes to reconnect Llagas Creek upstream of Silveira Lake with the natural channel by filling the Silveira Lake inlet channel. The Silveira Lake outlet, approximately 800 feet downstream from the West Little Llagas Creek diversion channel, would remain allowing water to enter the Lake as backwater from Llagas Creek during larger flows.

Llagas Creek around Silveira Lake would consist of approximately 1,550 feet of levee on the left overbank (looking downstream) from Olive Avenue to the West Little Llagas diversion channel. The levee will leave the existing channel undisturbed. The right overbank (looking downstream) consists of naturally high ground, except at the Silveira Lake inlet channel, and eventually ties into Silveira Lake. The average design channel velocity in this area is 4.2 feet per second.

The levee along the left overbank (looking downstream) would tie into the diversion channel levee. The minimum freeboard would vary from 3.5 to 2.0 feet. The levee will provide maintenance access to the area and consists of 1V:2H side slopes with a 12-foot top width, varying from 5 to 6 feet in height.

1.3 SCOPE OF WORK

The following scope of work has been performed to serve as a basis for the development of the opinions and recommendations presented in this draft report:

- Prepared Quality Control Procedures Manual.
- Reviewed available information including current levee cross sections, topographic drawings and design drawings (plan and profile sheets, Preliminary Plans dated March 2 and June 23, 2003) provided by the District.
- Reviewed proposed improvements including raised levee, channel toe excavation, flood walls, and retaining walls/gabions.
- Selected locations of new borings and piezometers (observation wells).

- Prepared Field Workplan.
- Cleared utilities at field exploration points through Underground Service Alert.
- Developed a Health and Safety Plan.
- Performed field exploration including:
  - 129 primary borings
  - 14 shallower borings drilled adjacent to 14 of the primary borings to obtain supplementary soil samples for laboratory testing
  - most primary boring depths were about 40 feet
  - 23 piezometers about 20 feet deep
- Prepared boring logs.
- Prepared discussion on area geology and seismicity.
- Prepared Phases I, II and III Laboratory Testing Plans.
- Performed laboratory testing, including:
  - Grain size distribution
  - Plasticity index
  - Unconfined compression, water content and dry density
  - Unconsolidated undrained triaxial compression
  - Consolidated undrained triaxial compression with pore pressure measurements
  - Consolidation
  - R-value
- Prepared Report of Field Activities.
- Performed preliminary geotechnical assessment.
- Identified key geotechnical cross-sections along channel.
- Characterized soil profiles for the 6 reaches.
- Identified potential geotechnical issues.
- Selected seventeen (17) most critical levee cross sections.
- Performed geotechnical calculations:
  - At seventeen cross sections
  - Factors of safety for slope stability for 4 loading conditions
  - Slope stability earthquake deformations
  - Bridge design parameters

- Floodwall design parameters
- Box culvert and fish barrier design parameters
- Settlement
- Seepage
- In accordance with Corps Engineer Manual No. 1110-2-1913 (Design and Construction of Levees) and Engineer Manual No. 1110-2-1904 (Soil Mechanics Design - Settlement Analysis)
- Liquefaction potential
- Reviewed and analyzed cross sections of channel in close proximity to existing structures at four (4) locations
- Structural pavement design at three (3) locations

- Prepared draft geotechnical report:
  - Summarized the results of all the above tasks
  - Developed geotechnical opinions and recommendations regarding the structural integrity of levees.
  - Developed geotechnical design parameters for the reinforced concrete flood walls
  - Developed geotechnical design parameters for the box culverts and bridge
- Attended 10 progress meetings.

1.4 PREVIOUS INVESTIGATIONS

Logs of borings are presented in Appendix A of Volume II for the projects discussed in this section.

The District performed two investigations for portions of the subject project in Reaches 7A and 7B. The results of those investigations were presented in two reports, including the following:

- "Field Activities Report, Box Culverts Within Reach 7B of the Upper Llagas Creek Flood Protection Project, Morgan Hill, California, Project No. 26174052-1217", dated April 19, 2006 (by District). Attached to that report was "Final Report, Geologic Investigation, Upper Llagas Creek Box Culverts – Reach 7B, Morgan Hill, California" dated September 25, 2003 (by URS). Seven borings were drilled by District on June 9 through June 13, 2003 including B-1 through B-7 (presently numbered SB 7B-1 through SB 7B-7). Piezometers were installed in Borings B-2, B-3, B-5 and B-7.
- "Engineering Geologic Evaluation, Llagas Creek Reach 7A, Morgan Hill, California" dated October 29, 2004 (by District). Thirteen borings were drilled by District on June 7 through June 11, 2004; nine were drilled for geotechnical reasons (Borings B7a-5 through B7a-13, i.e. SB 7A-5 through SB 7A-13) and four were drilled for hazardous materials testing

(Borings 7a-H1 through 7a-H4). Piezometers were installed in Borings SB 7A-9, SB 7A-10, SB 7A-11 and 7a-H4.

Pacific Geotechnical Engineering reviewed their company database for projects within close proximity of the project alignment. Based on their review, Pacific Geotechnical Engineering identified several project sites, as shown in the following table.

Boring Location	Data Description	Approx. Distance to Existing Channel (feet)	Year Drilled
Reach 4 – Ariki Property – 10025 Center Avenue	One 40 foot boring, testing includes PIs, UC, MD	175	Undated
Reach 6 – Llagas Avenue Warehouse, 13920 Llagas Ave.	One 30.5 foot boring, additional borings, tests include MD PP, - #200	210	1989
Reach 7A – Fire Station, Watsonville Road 767-23-017	One 21.5 foot boring, additional borings, tests include R, CT, MD, UC, PP, - #200	290	1989
Reach 7B – Edmundson Avenue Bridge over Llagas Creek (Pacific Geotechnical Engineering Project 1048E)	3 borings with solid flight augers deeper than 40 feet; 44.5', 76.5', 59.5'; testing includes UC, MD, PP	0	1968
PI = Plasticity Index MD = Moisture & Density UC = Unconfined Compression PP = Pocket Penetrometer CT = Compaction Test R = R-value - #200 = Percent Fines			

In addition URS obtained boring logs of other nearby projects, as summarized below.

Boring Location	Data Description	Approx. Distance to Existing Channel (feet)	Year Drilled
Reach 5 – Highway 101 Llagas Creek Bridge	4 borings, about 60 feet deep	0	1968
Reach 7A – West Middle Road Bridge	3 borings, 51½, 52½ and 71½ feet deep, MD, UC	0	1989
Reach 7A – Watsonville Road Bridge	3 borings, two at 51½ feet deep, one at 71 feet, MD, UC	0	1989
Reach 7B – Edes Court Box Culvert	3 borings, 2 at depths of 21 feet, 1 at 40.5 feet, no laboratory tests available	0	Undated

1.5 DATA REVIEW

As part of this geotechnical study, we reviewed the available preliminary levee design and other documents including the following:

- Draft Preliminary Plans for the Upper Llagas Flood Protection Project (plan and profiles) dated March 2, 2003 and June 23, 2003 (11 inch by 17 inch, 66 sheets total) by Corps.
- "Upper Llagas Creek Flood Control Project, Morgan Hill, California, Sediment Assessment and Hydraulic Design Office Report" dated December 2002, by Corps.
- "Engineering Geologic Evaluation, Llagas Creek Reach 7A, Morgan Hill, California" dated October 29, 2004 by District (James L. Nelson).
- "Preliminary Geotechnical Evaluation, Llagas Creek Watershed, Project – PL566, Santa Clara County, California," JS 13493, dated September 11, 1975 by Woodward-Clyde Consultants.

We also reviewed the following references in order to prepare the Health and Safety Plan for the field exploration program:

- "Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reaches 4, 5, 6 and 7B, Morgan Hill, California, Volume No. 1" dated December 16, 2003 by Piers Environmental Services, Inc.
- "Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reaches 4, 5, 6 and 7B, Morgan Hill, California, Volume No. 2" dated December 16, 2003 by Piers Environmental Services, Inc.
- "Report of Limited Phase II Soil Sampling for Selected Parcels along Upper Llagas Creek Flood Protection Project, Reaches 4 and 5, Gilroy, California" dated August 6, 2004 by Piers Environmental Services, Inc.
- "Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reaches 5 and 6, Gilroy and San Martin, California" dated November 27, 2002 by Piers Environmental Services, Inc.
- "Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reaches 7B and 4, Morgan Hill and Gilroy, California" dated July 24, 2002 by Piers Environmental Services, Inc.
- "Part 2 of 2, Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reaches 7B and 4, Morgan Hill and Gilroy, California" dated September 12, 2002 by Piers Environmental Services, Inc.
- "Phase II Hazardous Materials Investigation, Llagas Creek Reach 7A, Morgan Hill and San Jose, California" dated August 30, 2004 by Light, Air and Space Construction.
- "Phase I Environmental Site Assessment Report for Selected Parcels along Upper Llagas Creek, Reach 8, Morgan Hill, California" dated April 21, 2003 by Piers Environmental Services, Inc.

- "Phase I Environmental Site Assessment Report for Reaches 14, Selected Parcels along East Little Llagas Creek, Morgan Hill, San Martin and Gilroy, California" dated February 7, 2005 by Piers Environmental Services, Inc.

SECTION 2

2.1 GEOLOGIC SETTING

2.1.1 Physiographic Setting

The project area is located in the southeastern Santa Clara Valley, which lies between the Santa Cruz Mountains to the southwest and the Diablo Range to the northeast. The project area encompasses the upper reaches of Llagas Creek, a south-flowing drainage that constitutes one of the northerly headwaters of the Pajaro River drainage. The northern end of the project area lies just south of a drainage divide that crosses the floor of the Santa Clara Valley within a few miles north of the project area. As a generality, north of this point surface and groundwater flow northward into the Coyote Creek drainage, thence to San Francisco Bay. South of this point, surface and groundwater flow southward into the Pajaro River drainage, thence to Monterey Bay. On Corps and District maps, Reach 8 and part of Reach 7 (once the diversion channel is constructed) lie on West Little Llagas Creek. West Little Llagas Creek will join Llagas Creek at Lake Silveira; Reaches 6, 5, and 4 therefore lie on Llagas Creek. Reach 14 lies along East Little Llagas Creek, joining Llagas Creek at the downstream end of Reach 5.

2.1.2 Regional Geologic Setting

The portion of the Santa Cruz Mountains west of the project area is assigned to the New Almaden structural block by McLaughlin and others (2001). Bedrock in this area consists largely of Jurassic to Cretaceous-age Franciscan Complex bedrock assigned to various terranes based on their tectonic origin (see Figure 29, Regional Geologic Map). Individual rock types within the watersheds contributing sediment to the project area include greenstone, sheared greywacke sandstone, and minor limestone and serpentinite. Miocene rocks, including siliceous shale of the Monterey Formation, and unnamed sandstone, are locally exposed near the western valley margin but were not formed under the current tectonic regime.

The lower slopes of the Diablo Range east of the project area are primarily underlain by Plio-Pleistocene non-marine rocks that overlie Great Valley Sequence sedimentary rocks and Franciscan Complex metamorphic rocks (see Figure 29, Regional Geologic Map). Depending on clast provenance, these rocks are assigned to the Santa Clara Formation (fluvial boulder to pebble conglomerate, sandstone, siltstone and lesser mudstone deposited in a lacustrine environment), and the Silver Creek Gravels or Packwood Gravels (both as above plus volcanic clasts including tuff (McLaughlin and others, 2001; CGS, 2004). Pliocene basaltic volcanic rocks are also mapped in association with the Plio-Pleistocene sedimentary rocks.

The Santa Clara Valley floor is infilled primarily with Quaternary-age fluvial sediment within alluvial fans derived from the Santa Cruz Mountains and Diablo Range, and the valley-floor Llagas Creek fluvial system. The properties of this Quaternary sediment are the focus of this investigation.

Dibblee (1973a, 1973b) prepared a relatively early map of regional geology, emphasizing bedrock geology. This series of maps describes the alluvium along the Llagas Creek corridor as consisting of gravel, sand and clay.

Pacific Geotechnical Engineering (1994) prepared a folio of geologic maps and ground movement potential maps for the City of Morgan Hill at 1"=200' scale. The modern channel of

Upper Llagas Creek is mapped as being underlain by alluvium consisting of poorly- to well-consolidated gravel, sand, silt and clay.

Wentworth and others (1998) prepared a regional geologic map integrating detailed mapping in a regional framework. This map is used as the basis for Figure 29 (Regional Geologic Map), with the project reaches superimposed.

Knudsen and others (2000) prepared a regional map emphasizing Quaternary geology, along with a derivative map of liquefaction susceptibility. Their geologic map provides more detailed Quaternary interpretation than previous mapping efforts, and subdivides the Quaternary deposits on the basis of age and depositional environment. This map is used as the basis for Figure 30, Quaternary Geologic Map.

2.1.3 Tectonic Influence on Local Geologic Setting

The tectonic influence on local Quaternary geology is important to the project because it has governed the composition of sediment delivered to the Llagas Creek system, the ranges in grain size of sediment delivered to the system, and the evolution of drainage systems that now contribute runoff to the project area.

The project area lies near the southern end of the San Francisco Bay structural block, bounded on the west by the strike-slip San Andreas fault, and on the east by the Calaveras fault (Figure 31, Selected Active Faults). The intervening lowland, making up the southern Santa Clara Valley and including the project area, is arched upward in a broad NW-SE-trending antiform, the hingeline of which passes just southwest of the project area, through Uvas Valley (McLaughlin and others, 2000). This antiform is probably related to transfer of transpressional strain from the San Andreas to the Hayward/Calaveras faults via southwest-dipping thrust/reverse faults such as the Foothills fault system of range-front thrusts and their blind (buried) counterparts beneath the southern Santa Clara Valley (McLaughlin and others, 2000). Since Miocene time, bedrock in the Santa Cruz Mountains west of the project area has been undergoing tectonic uplift, generating topographic relief and a supply of sediment to the project area (Burgmann and others, 1994; McLaughlin and others, 2000).

The tectonic activity of the valley floor in the vicinity of the project area is evidenced by:

- The presence of a drainage divide crossing Quaternary deposits of the valley floor just north of the project area.
- The existence of an ill-defined basin west of Coyote Creek, north of this drainage divide and therefore north of the project area. This creek collects runoff from portions of the valley floor, but has not yet been fully integrated into the Coyote Creek drainage network (California Geological Survey [CGS], 2004).
- The position of Upper Llagas Creek tight up against the western margin of the valley; in places the western bank may consist of pre-Miocene bedrock under minimal cover, while the nearest exposed bedrock east of the Upper Llagas Creek lies along the eastern Santa Clara Valley margin 2 to 3 miles to the east.
- The capture of the Anderson Lake and Coyote Lake drainages by the northward-flowing Coyote Creek in Pleistocene time, in part as a result of movement on the Calaveras fault.

Previously, these drainages likely flowed south and west into the Pajaro River drainage, with at least part of these paleo-drainages passing through the project area.

- The presence of a warped, uplifted late Pleistocene fluvial terrace west of the project area that has apparently affected flow patterns of the Llagas Creek (Chesbro Reservoir and Paradise Valley) and upper Uvas Creek drainages (McLaughlin and others, 2000).

2.1.4 Alluvial Fan Systems/Drainages in the Project Area

Three major alluvial fan systems have contributed sediment to the project area, with clast types reflecting the bedrock type in their source areas. Other minor drainages also contribute sediment from local sediment sources. The main fan systems and sediment sources are:

- The Llagas Creek drainage (including Chesbro Reservoir and Paradise Valley) west of the project area; this drainage flows out of the Santa Cruz Mountains through the Silveira Lake area just NW of San Martin, thus contributing sediment to the project area. The clast types are primarily Franciscan Complex rock types. The project area lies approximately at the point that this sizeable watershed discharges from the range front (see Figures 29 and 30).

  This alluvial system may once have been even larger, and may have included the upper portion of the Uvas Creek watershed before late Pleistocene time. McLaughlin and others (2000) describe 20m (66 feet) of uplift that occurred since approximately 50,000 years ago, based on a deformed late Pleistocene fluvial surface that forms a divide between these two watersheds. The uplift now forces runoff from this upper portion of the modern Uvas Creek drainage to flow southward; before this time at least some of the upper Uvas Creek watershed may have drained eastward into Llagas Creek.

  Within the project area, Llagas Creek flows within a corridor mapped as being underlain by latest Pleistocene and Holocene alluvium, across an alluvial surface considered to be late Pleistocene and Holocene (see Figure 30).

- The Anderson Lake watershed in the Diablo Range now discharges northwestward into Coyote Creek, thence toward San Francisco Bay. However, it is apparent from topography that a large, well-developed conical alluvial fan extends completely across the Santa Clara Valley to the western margin, and south past Dunne Avenue (see "Coyote Creek fan" on Figure 29). This fan has helped to keep Llagas Creek forced against the western margin of the valley. The clasts in this fan are derived from a diverse source area with primarily Cretaceous and Tertiary marine and non-marine sedimentary rocks. While the modern drainage network on this fan surface is by tiny, unnamed drainages, the fan as a whole was constructed by much more robust flows similar in scale to today's Coyote Creek, and thus the fan contains coarser sediment than today's much smaller creeks would indicate.

  Apart from the currently-active, northward flowing portion of the fan associated with the modern Coyote Creek, the Coyote Creek fan is mapped as a Pleistocene deposit.

- Minor westward-flowing creeks draining westward out of the Diablo Range into the project area include San Martin Creek, Center Creek, New Creek, Rucker Creek and Skillet Creek. The headwaters of Church Creek are in a dissected Pleistocene and older (Knudsen and others, 2000) alluvial fan built up by a major drainage system that once included the entire Coyote Lake watershed. This fan, the toe of which reaches the project area, also contains coarser sediment than the current small creeks would indicate.

2.1.5 Quaternary Deposits Mapped in the Project Area

The most comprehensive Quaternary mapping to date that encompasses the project area is Knudsen and others (2000), and a portion of this map is used as the base for Figure 30. The same authors prepared a liquefaction susceptibility map that also constitutes the most recent and comprehensive evaluation to date for the entire project area (see Figure 32, Liquefaction Susceptibility Map). The liquefaction mapping is discussed later in a separate section.

For reference, the Quaternary deposits mapped as underlying various reaches are briefly reviewed below, with primary reference to Knudsen and others (2000) and Pacific Geotechnical Engineering (1994). This mapping, together with the project reaches, is shown on Figure 30.

- Reach 8 - Beginning at the northern end of the project area, near the northern limit of Morgan Hill, the existing creek channel (West Little Llagas Creek on District maps) is mapped by Knudsen and others (2000) as underlain by latest Pleistocene to Holocene alluvial fan deposits (Qf). The same area is mapped by Pacific Geotechnical Engineering (1994) as Quaternary alluvium (Qa) incised into older alluvium (Qoa). South of about West 1st Street, the channel's course is forced eastward by a prominent hill underlain by Franciscan Complex greenstone. Between 3rd and 4th Streets, the channel approaches within tens of feet of the lower hillslopes, before swinging away from the bedrock hillfront south of about 5th Street.
- Reach 7 - South of approximately Ciolino Avenue (which lies within Reach 7B), the channel alignment is mapped by Knudsen and others (2000) as lying in Holocene alluvial fan deposits (Qhf) only as far south as approximately Edes Court; south of this point it flows across Pleistocene alluvial fan deposits (Qpf) to approximately the Tennant Avenue bridge, then in latest Pleistocene to Holocene alluvial fan deposits (Qf) until reaching the confluence near Lake Silveira. URS (2003) called attention to Holocene alluvial fan deposits mapped by Knudsen and others (2000) between approximately Spring Ave. and Ciolino Ave.

  Pacific Geotechnical Engineering (1994) maps this entire reach as being underlain by Quaternary alluvium (Qa) incised into older alluvium (Qoa), past the confluence with Edmundson Creek (Little Llagas Creek on USGS quadrangle) south of La Crosse Drive. South of La Crosse Drive, the existing and proposed channels diverge, and are both mapped as Quaternary alluvium (Qa) incised into older alluvium (Qoa).

  The proposed diversion channel would continue south to a confluence with the main stem of Llagas Creek, south of West Middle Avenue, at Lake Silveira. At this point, Llagas Creek is again forced eastward by a second bedrock high, also mapped as Franciscan Complex greenstone (Pacific Geotechnical Engineering, 1994). Lake Silveira owes its existence to quarrying of the relatively coarse alluvial gravels deposited in this reach. The alluvial fan of Llagas Creek is large enough that the natural course of West Little Llagas Creek is forced to take an eastern course that lacks a well-developed confluence with Llagas Creek – one of the factors in local flooding.
- Reach 6 – Southeast of Silveira Lake, the Llagas Creek channel alignment is mapped (Knudsen, 2000) as lying within a belt underlain by Holocene alluvial fan levee deposits (Qhl). The width of this belt approximately corresponds to an envelope fitting or enclosing creekbed meanders. Latest Pleistocene to Holocene alluvial fan deposits (Qf) are mapped to the east and west of this belt.

- Reach 5 – This short reach is mapped (Knudsen and others, 2000) as underlain by latest Holocene alluvial fan deposits (Qhfy) and Holocene alluvial fan levee deposits (Qhl).
- Reach 14 – This reach consists of East Little Llagas Creek (on District maps), and collects the flow of Corrallitos Creek, San Martin Creek, Center Creek, New Creek and Church Creek. The channel is mapped as underlain by latest Holocene alluvial fan deposits (Qhfy) incised into or flowing around the western perimeter of the dissected Pleistocene and older alluvial fan with its apex near the headwaters of the modern Church Creek.
- Reach 4 – Llagas Creek in Reach 4 is mapped (Knudsen and others, 2000) as lying within a belt underlain primarily by Holocene alluvial fan levee deposits (Qhl). Sediments outside of the belt are mapped as Holocene alluvial fan deposits (Qhf).

2.1.6 Independent Aerial Photographic Evaluation of Regional Mapping

In order to evaluate the regional Quaternary geologic mapping of Knudsen and others (2000), we examined the earliest available vintage of aerial photographs providing high-quality stereoscopic coverage for the project area – photos taken in October of 1939 (see References). These photos show conditions at a time when Morgan Hill and San Martin were small towns, and land use was overwhelmingly agricultural. Most importantly, drainage patterns had only been altered to a fairly limited extent by local farmers, so natural drainageways and their soil/geomorphic associations are relatively unobscured.

Our judgment is based on a limited evaluation that encompasses primarily the immediate project area, but we identified no areas where we would draw significantly different lines than shown on Knudsen and others (2001). Their map was digitized at a scale of 1:24,000 (Knudsen, personal communication), and published without an underlying road or topographic base, so registration of our interpretive mapping to the published linework is inexact.

One general observation is that the complexity/intricacy of geologic contacts is more intricate than can be reasonably shown on a 1:24,000 scale map, and there are likely similarly complex relationships in the subsurface. Secondly, there is likely a continuum of ages for the geomorphic surfaces (e.g. alluvial fan surfaces), with sharp definition of Holocene from Pleistocene deposits necessarily being a somewhat arbitrary decision.

2.1.7 Implications of Subsurface Findings for Quaternary Mapping

In order to evaluate how well the regional geologic mapping (primarily Knudsen and others, 2000) correlated with materials encountered in our borings, we cross-compared the two. The regional mapping discussed above indicates that the reaches that are most likely to be underlain at relatively shallow depths by Pleistocene deposits, which tend to be relatively well-consolidated, include: 1) downtown Morgan Hill in Reach 7 from about Edes Court to the Tennant Avenue bridge (roughly between Stations 108+00 and 97+00); and 2) Reach 14 north of San Martin Creek.

We found that the materials encountered and the blow counts observed generally support the mapped Pleistocene deposits in these areas. The borings indicate, however, that relatively well-consolidated materials are much more widespread than this mapping would suggest. It may be that the regional geologic inferences regarding ages of alluvial fan surfaces are valid, but that they are not reflected by profound differences in properties of the earth materials.

Selected geologic observations regarding trends in materials encountered for the various reaches are reviewed briefly below. Any fill present is not discussed.

Reach 4 – Fine-grained deposits (silt or clay) are found in the upper 0.6 to 2.4m (2 to 8 feet) in four contiguous borings (SB 4-2, 4-3, 4-4, 4-5). This is consistent with generally fine-grained, geologically recent deposits mapped along the valley floor.

Gravel forms much or all of the bottom of three borings (SB 4-6, 4-7, 4-8), with the same interval also apparent in two nearby borings (SB 4-4, 4-9). This could be interpreted as either an old (probably Pleistocene) channel complex, or as part of the Pleistocene alluvial fan mapped nearby to the east.

Reach 5 – This reach essentially bridges the interval between Llagas Creek and East Little Llagas Creek.

Fine-grained deposits (clay) are observed at the bottom of the westernmost boring (SB 5-1); this interval thins eastward.

Gravel is concentrated in two borings (SB 5-3, 5-4).

There is no gravel at all in the easternmost boring. The variability in this reach probably reflects shifting channel location.

Reach 6 – Gravel is common throughout this reach, especially in the northern part that lies just downstream of the Llagas Creek fan. The proximity to the source and to areas of tectonic uplift probably results in most gravel being deposited in the northern part of this reach.

Fine-grained deposits are increasingly common at the bottom of the borings south of SB 6-8, and are probably correlative with a similar fine-grained interval in the bottom of some Reach 5 borings. The origin of these deposits is unclear.

Reach 7 – Fine-grained deposits (clay) are dominant in northern part of Reach 7A. This reach lies in the currently poorly drained interval between West Little Llagas Creek and Llagas Creek – there is no well-defined connection now, but the proposed diversion channel will link these two. The borings' location off of the existing channel trend is consistent with their containing a greater proportion of fine-grained overbank deposits. This reach also lies off the northern margin of the Llagas Creek fan, which contributes to low stream gradient.

Gravels are increasingly represented to the south, reflecting proximity to the modern Llagas Creek fan. In general, gravels lie below about 1.2 to 6.1m (4 to 20 feet) of finer-grained (sand, silt, clay) material. The gravels may represent a channel complex that has shifted laterally over time, with the upper finer-grained interval representing overbank deposits.

Reach 8 – The northern end of this reach is very dominantly clay with minor sand and silt, as far south as SB-8A-4F. The northern end of this reach lies in the headwaters region of West Little Llagas Creek, near the drainage divide. There are no nearby major creeks to supply coarse sediment. The materials encountered appear consistent with the geology as mapped.

Gravel and sand increase in proportion south of SB-8A-4F, with gravels typically buried below roughly 1.8 to 3.7m (6 to 12 feet) of finer (sandy) sediment. The shift southward to gravels south of SB-8A-4F may represent the Anderson Lake fan, mapped as a Pleistocene fan.

Reach 14 – The northernmost two borings in this reach lack any gravel, which is consistent with their position between two Pleistocene alluvial fans.

Gravels are found south of this point, and the generally high blow counts are consistent with deposits of the Pleistocene alluvial fans mapped in this area.

2.2 REGIONAL FAULTS AND SEISMIC SOURCES

No active faults are mapped as passing through the project area (see Figure 31, Selected Active Faults).

The two major seismic sources of most relevance to the project are the Calaveras fault and the San Andreas fault, from the standpoints of proximity to the project area, seismic capability, and fault activity. Other seismic sources contribute to a lesser degree to seismic hazard in the project area. In terms of the amount of energy released by a given earthquake, and in terms of applicability to project design, moment magnitude (M_w) is probably the appropriate magnitude scale as opposed to the Richter magnitude scale, since M_w is related to the area of rupture on a fault surface, and Richter magnitude is a more indirectly derived term. However, M_w is a relatively recent term, and older discussions of historic earthquakes tend to employ Richter magnitude, particularly for sparsely instrumented or non-instrumented earthquakes. Richter magnitude values are numerically typically slightly higher than M_w values.

2.2.1 Calaveras Fault

The Calaveras fault passes through the lower foothills of the Diablo Range and roughly forms the eastern margin of the southern Santa Clara Valley (Jennings, 1994). The 120-km-long Calaveras fault is differentiated into the northern, central and southern segments, with the predominant sense of movement being right-lateral strike slip. The northern segment, which extends from Calaveras Reservoir to about Danville, may currently be locked and little seismicity is observed along it (WGCEP, 2003). Seismicity defines central and southern segments of the fault south of Calaveras Reservoir, with the southern and central segments undergoing creep. The creeping southern segment merges with the San Andreas fault near Hollister.

The Calaveras fault has generated a number of moderate magnitude, damaging earthquakes during historic time: 1897, 1911, 1979, and 1984. The magnitudes for all four of these earthquakes were nearly identical: 1897 – M_w (estimated) 6.2; 1911 – M_w (estimated) 6.1; 1979 – M_w 5.9; and 1984 – M_w 6.2 (WGCEP, 2003). Current research (CGS, 2003) indicates that the maximum earthquakes for the northern, central and southern segments of the Calaveras fault are M_w 6.8, 6.2, and 5.8 respectively. The Working Group on California Earthquake Probabilities (WGCEP) (2003) considers a combined rupture of the southern and central segments to have a potential M_w 6.4.

The project area lies between approximately 5 and 6km west of the Calaveras fault, depending on the individual reach.

2.2.2 Sargent Fault

The Sargent fault is a steeply southwest-dipping reverse oblique fault that links the San Andreas fault near Lake Elsman, and the Calaveras fault just northwest of Hollister. The oblique component of offset is right-lateral, based on offset of geomorphic features and ("last motion") slickensides (Bryant and others, 1981). Its total length is approximately 55km (Jennings, 1994).

There is evidence for creep along the Sargent fault. Prescott and Burfurd (1976) measured 3 +/- 1mm/yr of creep in the southern third of the fault. The Sargent fault experienced sympathetic triggered slip during the 1989 Loma Prieta earthquake (Aydin and others, 1992).

McLaughlin and others (2000) consider the Sargent fault to represent the southwesternmost and structurally highest of a suite of thrust or reverse faults of Neogene age that includes the Hooker Gulch, Berrocal, Shannon, and Monte Vista faults. In general, the timing of inception of movement on these faults becomes younger to the northeast, as does their activity in general. Like other east-vergent thrust faults east of the San Andreas fault, the Sargent fault is generally thought to be tectonically coupled with the San Andreas fault at depth. Nolan and others (1995) suggest that the Sargent fault may not be tectonically coupled with the San Andreas, and that movement may be associated with distributed shear across the region. The fault database for the statewide probabilistic seismic hazard analysis (PSHA) developed by CGS (2003) treats the Sargent as a vertical fault.

The CGS (2003) has deleted the Sargent fault from their probabilistic model, although they consider it to be capable of a maximum earthquake of M_w 6.8. The Sargent fault lies approximately 10km west of the project area.

2.2.3 San Andreas Fault Zone

The San Andreas fault is the dominant fault in California, extending a total of approximately 1,200km from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in Northern California. As it passes through the greater Bay Area, it runs from beyond Pt. Reyes to the north, down the San Francisco Peninsula, and extending on beyond Hollister to the south. This fault has generated at least four large, damaging earthquakes during historic time: 1838, 1857, 1906 and 1989. In addition, an 1836 earthquake once considered to have occurred on the Hayward fault is now thought to have occurred south of Loma Prieta in the Santa Cruz Mountains on an unknown fault (Toppozada and Borchardt, 1998). The earthquake of 1838 probably caused ground rupture from San Juan Bautista to San Francisco, and was centered somewhere in between; it had an estimated moment magnitude of about 6.8 (Bakun, 1999) to 7.5 (Toppozada and Borchardt, 1998). The earthquake of 1857 occurred in San Luis Obispo County; it had an estimated Richter magnitude of approximately 8.0. The 1906 earthquake was probably centered just offshore of the Golden Gate Bridge of San Francisco Bay, and had an estimated moment magnitude of approximately 7.9. It ruptured a distance of approximately 475km, from approximately San Juan Bautista at the southern end, to Punta Delgada on the northern end (Lawson, 1908). The 1989 Loma Prieta earthquake was epicentered in the Santa Cruz Mountains. This M_w 6.9 earthquake caused 64 deaths, about 4,000 injuries and about 6 billion dollars of damage in the Bay Area.

The CGS and WGCEP currently consider the maximum earthquake for the Peninsula segment of the San Andreas fault to be approximately moment magnitude 7.1 (CDMG, 1996; CGS, 2003;

WGCEP, 2003). The maximum earthquake for the Santa Cruz Mountains segment is considered to be moment magnitude 7.0. Both segments were considered by the CGS (1996) to have the same 400-year return intervals for the maximum earthquake, although more recent work suggests a shorter return interval (e.g. Hall and others, 1999). The revised CGS probabilistic model for the state and the WGCEP's probabilistic model consider the Santa Cruz Mountains segment of the San Andreas fault to have a maximum earthquake of M_w 7.0 (CGS, 2003; WGCEP, 2003).

The San Andreas fault lies approximately 16 to 17km west of the project area, depending on project reach.

2.2.4 Hayward Fault

The Hayward fault forms the eastern margin of the San Francisco Bay basin. The last major earthquake on the Hayward fault occurred in 1868 along a "southern segment" of the fault, and had an estimated Richter magnitude of 7.0. Until recently, it was thought that a similar-magnitude earthquake in 1836 occurred on a northern segment of the fault. However, as noted above, recent research suggests that the 1836 earthquake occurred on a different fault (Toppozada and Borchardt, 1998), and that the Hayward fault may be unsegmented. The CGS (2003) considers the maximum earthquake for the southern Hayward segment to be M_w 6.7. As defined by California Division of Mines and Geology (CDMG) 1998, the southern end of the Hayward fault seismic source is considered to lie 35km north of the northernmost part of the project area.

At depth, the distribution of seismicity indicates a relatively simple linkage between the southern end of the steeply east-dipping Hayward fault, and the Calaveras fault, into which it roots (Ponce and others, 2005).

The surficial expression of this transfer zone is complex, however, and at the ground surface the Hayward fault feathers out southeastward into a multitude of strands (Wentworth and others, 1998) as strain is transferred to the Calaveras fault to the east. The intervening terrain between these two major faults is characterized by areas of uplift (e.g. Mission Peak) with extensive mass-wasting. The activity of the multitude of small fault strands is under debate, as some structures previously attributed to tectonism have been re-attributed to mass-wasting (Fenton and Hitchcock, 2001).

The WGCEP (2003) and the CGS (2004) probabilistic models no longer differentiate this transition zone (Hayward fault "southeast extension") from the southern segment of the Hayward fault. As defined by CDMG (1998), the southern end of the Hayward fault southeast extension is considered to lie 17.7km north of the northernmost part of the project area.

2.2.5 Foothills Thrust Belt (Monte Vista-Shannon Seismic Source)

This seismic source essentially composites several separately mapped frontal thrust faults along the northeastern margin of the Santa Cruz Mountains. McLaughlin and others (2000) consider the complexly interlaced Berrocal, Shannon and Monte Vista faults as one fault system. Together with the structurally higher Hooker Gulch and Sargent faults, these constitute a suite of range-front thrust faults that are Miocene and younger. Since approximately Miocene time, these faults have accommodated transpression that cannot be taken up by the major strike-slip faults of the region.

An estimated Richter magnitude 6.8 earthquake experienced by the San Francisco Bay region in 1865 has been tentatively attributed to the Monte Vista or Shannon fault, based in part on distribution of reported shaking intensity (Tuttle and Sykes, 1992; Yu and Segall, 1996). As defined by CDMG (1998), the southeastern end of the Monte Vista/Shannon seismic source is considered to lie 20.1km north of the northernmost part of the project area.

While some of these west-dipping faults are not considered to be independent seismic sources, this seismic source as an aggregate is considered capable of a M_w 6.7 earthquake (CGS, 2003). McLaughlin and others (2000) calculate from the relationships of Wells and Coppersmith (1994) that if this fault zone were to rupture from the maximum depth of seismicity (8 to 10km) to the surface all along its length of 30 to 40km, an M_w 6.6 to 7.0 earthquake could result.

2.2.6 Other Faults

2.2.6.1 Monterey Bay/Tularcitos fault

This fault lies essentially within Monterey Bay, and accommodates some of the right-lateral slip carried solely by the San Gregorio-Hosgri fault in areas farther north, in addition to reverse motion. This fault is considered capable of a M_w 7.3 earthquake (CDMG, 1996; CGS, 2003), and is included in the CGS statewide probabilistic seismic hazards model (CGS, 2003).

2.2.6.2 San Gregorio-Hosgri fault

The San Gregorio-Hosgri fault is a part of the San Andreas fault system, and is located offshore of Monterey Bay. Farther north, the fault comes onshore at Ano Nuevo near the San Mateo/Santa Clara County line, and is transitional northward to the Seal Cove fault. The northern portion of the fault has a relatively high slip rate of 7mm/yr (+/- 3mm) and is considered to have a maximum earthquake of M_w 7.5, while the southern portion is capable of a M_w 7.0 (CDMG, 1996; CGS, 2003). All faults in this system are considered seismically active.

2.2.6.3 Zayante-Vergeles fault

The Zayante-Vergeles fault accommodates strike-slip and reverse motion. It lies largely parallel to and west of the San Andreas fault in northern San Benito, Monterey, and southern Santa Cruz Counties. The CGS (2003) considers it to be capable of a M_w 7.0 earthquake.

2.2.6.4 Frontal thrusts, Eastern margin of Santa Clara Valley

Transpressional shortening is being accommodated by a set of "East Valley thrusts" (Fenton and Hitchcock, 2001). The capability of these faults to generate damaging earthquakes is under debate (Fenton and Hitchcock, 2001), and it is not treated as a seismic source by the CGS (2003). At least some of the structures mapped as faults appear to be accommodating mass wasting, although they may have originated as tectonic structures.

2.3 REGIONAL GEOLOGIC/SEISMIC HAZARD MAPPING

Previous regional geologic hazard evaluations encompassing all or part of the project area include Rogers and Williams (1974), Pacific Geotechnical Engineering (1994), Santa Clara County (2002 and web updates), Knudsen and others (2000), and CGS (2004). The findings of these regarding geologic hazards, excepting liquefaction, are reviewed briefly below. Liquefaction mapping is then reviewed by itself.

Rogers and Williams (1974) prepared an early evaluation of geologic hazards for the entire Santa Clara County, subsequently superseded by Santa Clara County (2002 and web updates).

Woodward-Clyde Consultants (1976) prepared a preliminary evaluation of geologic hazards in the portion of the Llagas Creek watershed approximately corresponding to the project area. Accompanying graphics for this letter report were not available, however the text indicates that several areas were identified as having a high potential for liquefaction, lurching and lateral spreading, and one area with a potential for differential settlement.

Pacific Geotechnical Engineering (1994) prepared a detailed geologic map and derivative ground movement potential map for the City of Morgan Hill. A review of 200 borings and records of highest recorded historic groundwater were used to generate a map of liquefaction potential. No areas with higher than a "low" liquefaction potential were identified; those areas near the project area are discussed below.

More recently, Knudsen and others (2000) evaluated liquefaction potential at a regional scale for a nine-county area including the project area. This map was used in 2004 as the basis for a public-information map widely distributed by ABAG (Association of Bay Area Governments); readers were referred to Knudsen and others (2000) for additional information. This mapping is discussed below.

Santa Clara County (2002 and subsequent web updates) prepared geologic and seismic hazard maps for the County that supersedes the Rogers and Williams (1974) maps, although some of the linework was adopted wholesale. None of the project area lies within a fault rupture hazard zone, or a landslide hazard zone established by the County.

CGS (2004) prepared a Seismic Hazards Zone map for the Morgan Hill 7.5-minute quadrangle, which encompasses only the part of the project area north of approximately West Dunne Avenue, in Reach 8. None of the project area is contained with a zone of required liquefaction investigation, based in part on the degree of consolidation of the Pleistocene-age Anderson Lake fan deposits, and their fines content.

Preparation of Seismic Hazards Zone maps for the quadrangles encompassing the remainder of the project area (Gilroy and Mt. Madonna 7.5-minute quadrangles) is underway (K. Knudsen [CGS], verbal communication 1/26/06) but no projected date for completion of a draft has been yet set. For the project area, no significant changes in Quaternary mapping are anticipated from the geology as mapped by Knudsen and others (2000), based on preliminary compilation and limited new mapping (Knudsen, verbal communication, 1/26/06).

2.3.1 Reach-By-Reach Previous Liquefaction Mapping in Project Area

Several compilations have been prepared in an effort to comprehensively identify areas of past liquefaction, as reviewed in CGS (2004); these include Knudsen and others (2000), Tinsley and others (1998), and Youd and Hoose (1978). CGS (2004) reports that the nearest documented locality of historic liquefaction reported in the Morgan Hill quadrangle lies approximately 14km north of the project area (Site 159 of Youd and Hoose (1978); site also portrayed on Knudsen and others (2000). This liquefaction occurrence dates from the 1906 earthquake on the San Andreas fault, and was described as consisting of ground cracks and ejected material in the bed of Coyote Creek.

As discussed above, the most comprehensive and recent geologic mapping and liquefaction susceptibility mapping are by Knudsen and others (2000); that Quaternary mapping is used as the basis for Figure 30, and the liquefaction susceptibility mapping forms the basis of Figure 32. More detailed mapping efforts exist (such as Pacific Geotechnical Engineering, 1994; and CGS, 2004), but they address only a limited portion of the project area.

Liquefaction susceptibility mapping is reviewed below on a reach-by-reach basis, with reference to the Quaternary deposits mapped as underlying each reach.

- <u>Reach 8</u> - Knudsen and others (2000) consider this reach to have generally *low* liquefaction susceptibility (on a five-class scale). Restricted areas of moderate liquefaction susceptibility are shown, which correspond to limited areas of Holocene alluvial fan deposits. CGS (2004) includes only the part of the project area north of about West Dunne Avenue in their map; none of this area is contained with a zone of required liquefaction potential, based in part on the degree of consolidation of the Pleistocene-age Anderson Lake fan deposits, and their fines content. Pacific Geotechnical Engineering (1994) prepared liquefaction potential maps based on historic high groundwater occurrence, and a review of about 200 boring logs. No liquefaction potential was identified in Reach 8.

- <u>Reach 7</u> - Knudsen and others (2000) consider most of this reach to have low liquefaction susceptibility. The area between approximately Ciolino Avenue and Edes Court that is underlain by Holocene alluvial fan deposits (Qhf) is mapped as having moderate liquefaction susceptibility. In contrast, Pacific Geotechnical Engineering (1994) show an area of low liquefaction potential a few hundred feet west of the channel, between approximately West Dunne Avenue and Spring Avenue, but none along the existing channel in this area.

  The southernmost portion of Reach 7, within approximately 350m north of Lake Silveira, is mapped by Knudsen and others (2000) as having high liquefaction susceptibility. County of Santa Clara (2002 and 2003 rev.), which took Knudsen and others' mapping into account in their 2002 revision, shows essentially the same area as lying within a liquefaction hazard zone. Pacific Geotechnical Engineering (1994) classify the portion of Reach 7 lying south of approximately La Crosse Drive as having a low liquefaction potential.

- <u>Reach 6</u> –Knudsen and others (2000) consider this reach to have moderate liquefaction susceptibility.

  County of Santa Clara (2002 and 2003 rev.) has established a thin (on the order of 50m wide) liquefaction hazard zone along the existing channel, from the outlet of Lake Silveira to the Church Avenue percolation ponds. The map location of this zone does not correspond exactly to the reach alignment shown on SCVWD maps; we infer that the zone location is based on aerial photographic interpretation of where the natural channel lay. Interestingly, a broad liquefaction hazard zone is established between, but distinct from, thin corridor-style liquefaction hazard zones established along Llagas Creek and East Little Llagas Creek.

- Reach 5 –Knudsen and others (2000) consider this reach to lie at the contact between areas having a moderate (south side of Reach 5) to high (north side of Reach 5) liquefaction susceptibility.

  Santa Clara County (2002) includes all of Reach 5 except the portion passing under Highway 101 within a liquefaction hazard zone.

- Reach 14 – Knudsen and others (2000) consider this entire reach to have moderate liquefaction susceptibility.

  Santa Clara County (2002) established a narrow liquefaction hazard zone for a meandering, presumably pre-development, stream course through this reach. This stream course has since been straightened and channelized, with the result that the current alignment only locally intersects the hazard zone, in perhaps 4 or 5 spots.

- Reach 4 –Knudsen and others (2000) consider this entire reach to have moderate liquefaction susceptibility, with areas of high liquefaction susceptibility mapped northeast of the channel.

  Santa Clara County (2002) established a narrow liquefaction hazard zone along this entire reach, with an additional broad area east of the existing channel also included in the hazard zone.

We preliminarily evaluated whether the mapped proximity of relatively consolidated Pleistocene alluvial fan deposits to the project area correlated with lower liquefaction potential. The regional mapping indicates that the reaches most likely to be underlain at relatively shallow depths by relatively consolidated Pleistocene deposits are: 1) within downtown Morgan Hill in Reach 7 from about Edes Court to the Tennant Avenue bridge; and 2) Reach 14 north of San Martin Creek. However, even in these areas, the complexity of the interrelationship between older and younger deposits, and the internal variability of the deposits is such that we did not observe a strong correlation. Please refer to Section 10 for results of liquefaction analysis that we have completed for the specific conditions encountered in the explorations made for the project.

2.4 GROUND MOTIONS IN PROJECT AREA

2.4.1 Deterministic Ground Motion Analysis

The deterministic approach that we used in this study involves the following steps:

- Identification of the potential seismic sources that could affect the site and estimation of their Maximum Credible Earthquake (MCE) that could reasonably be expected from these sources.
- Calculation of the peak horizontal ground accelerations (PGAs) that are likely to occur at the sites due to the MCE for each seismic source.

The first step, which is required in any assessment of earthquake hazards, requires a characterization of all significant seismic sources that could produce ground motions of engineering significance at the site. Required parameters include fault location, geometry, and orientation; sense of slip; and maximum magnitude.

2.4.2 Attenuation Relationships

To characterize the ground motions at the fifteen (15) Upper Llagas Creek sites as shown on Figure 33, we used empirical attenuation relationships to predict peak ground accelerations. We used three relationships to account for epistemic uncertainty. The relationships were selected on the basis of the site conditions and the tectonic environment.

The following table lists the three selected relationships along with their magnitude and distance definitions and limits of applicability. The site conditions assumed for each relationship are also listed in the table. For the Abrahamson and Silva (1997) and Sadigh et al. (1997) attenuation relationships, we used the soil site condition. For the Boore et al. (1997) attenuation relationship, we used an assumed shear wave velocity of 310 meters per second (1,000 feet per second), which is their recommended value for typical soil sites. The calculated peak ground acceleration values were not adjusted for site-specific variations in soil stiffness, but more detailed analyses could be performed for design of levee improvements.

Selected Attenuation Relationships

Attenuation Relationship	Definitions		Limits of Applicability		Site Condition
	Magnitude	Distance	Magnitude	Distance	
Abrahamson and Silva (1997)	M_w[1]	R_{rup}[2]	(see note 4)	(see note 4)	Soil
Sadigh et al. (1997)	M_w[1]	R_{rup}[2]	$4.0 \leq M_w \leq 8+$	$R_{rup} \leq 100$ km	Soil
Boore et al. (1997)	M_w[1]	R_{jb}[3]	$5.5 \leq M_w \leq 7.5$	$R_{jb} \leq 80$ km	V_s=310 m/s

Note:
1 = Moment magnitude.
2 = Closest distance to rupture surface.
3 = Closest horizontal distance to vertical projection of rupture surface.
4 = Not stated by the authors of the relationship; assumed applicable up to M_w8+ based on range of data used for its development.

The selected attenuation relationships were weighted equally for calculating the weighted average of peak ground accelerations.

2.4.3 Geologic Site Conditions

The sites are located on deep alluvial sediments. The reference site condition of the site was assumed as firm soil with a shear wave velocity of 310m/s (1,017 feet/second).

2.4.4 Results

The calculated median and 84th percentile values of PGAs, for Upper Llagas Creek Site Numbers 4-A through 14-O, are presented in Tables 1 through 15. For each point, the controlling seismic source is a M_w 7.1 earthquake along the Calaveras fault. This event is capable of generating a median PGA of 0.39g to 0.49g and an 84th percentile PGA of 0.62g to 0.77g at Site Numbers 4-A through 14-O.

2.5 GEOLOGIC/SEISMIC HAZARDS IN PROJECT AREA

2.5.1 Fault Ground Rupture

Fault ground rupture tends to occur along and in close association with existing fault traces. The potential for future fault ground rupture is highest along fault traces which have undergone geologically recent (Holocene) movement. No active fault traces are identified within the project area, and we judge the potential for fault ground rupture to be very low.

2.5.2 Liquefaction

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to the cyclic loading associated with earthquake ground shaking. In extreme cases, the soil particles can be suspended in groundwater, resulting in the deposit becoming mobile and fluid-like.

The potential for liquefaction is addressed in Section 10, on the basis of project-specific subsurface exploration, laboratory testing, ground-motion estimation, and engineering analysis. In general, we believe the primary consequence of liquefaction will be post earthquake ground settlement. Details are presented later in Section 10.

2.5.3 Lateral Spreading and Lurch Cracking

Based on the results of the liquefaction analysis and post-liquefaction slope stability analysis described in Section 10, it appears that lateral spreading and lurch cracking are unlikely. However, this opinion is based on a limited number of sections analyzed for post-liquefaction slope stability.

2.5.4 Other Seismically Induced Ground Deformation

Apart from the potential for lateral spreading and localized calving or sloughing of banks, we judge the potential for seismically induced landsliding to be generally low from a geologic standpoint. More detailed analysis of bank stability is provided in Section 4.

2.5.5 Seiche

Seiche is a phenomenon involving the "sloshing" or rhythmic oscillation of a body of water in response to earthquake ground shaking; the associated hazard has to do with the potential for inundation by water escaping its natural confines (e.g. reservoir, lake, bay, etc.). In our judgment, none of the bodies of water associated with the project area (i.e. Lake Silveira, Church Avenue percolation ponds) has a potential to generate a significant seiche.

2.5.6 Tsunami

The project area is not located within a coastal area potentially affected by a tsunami.

2.5.7 Seismically-Induced Dam Failure Inundation

The closest sizeable impoundments of water upstream of the project area are Anderson Reservoir and Chesbro Reservoir. These reservoirs are of such a size that their operation is subject to state oversight, in part to guard against seismically-induced inundation by dam failure.

2.5.8 Other Geologic Hazards

2.5.8.1 Landsliding

No landslides are mapped along the project reaches (Nilsen, 1972; Pacific Geotechnical Engineering, 1994). Due to the project's valley-floor setting, the potential for landsliding is very low, apart from localized sloughing of oversteepened banks. Bank stability is considered in the engineering analysis.

2.5.8.2 Volcanism

No Quaternary-active volcanoes are mapped within hundreds of miles of the site (Jennings, 1994), and the potential for volcanism to affect the project area is judged to be very low.

2.6 SUMMARY

There are no known active faults that cross the project alignment. However, the levees will be subject to strong ground shaking from known nearby active faults.

SECTION 3

3.1 SURFACE CONDITIONS

3.1.1 Reach 4

The Reach 4 alignment follows the existing Llagas Creek alignment through agricultural land between Buena Vista Avenue and the confluence with Reaches 5 and 14 (north of Masten Avenue). In general, there are a number of scattered one-story houses and farm buildings set back at relatively moderate to large distances from the proposed channel. The ground surface drops in a southerly direction from about Elevation 245 to Elevation 219 (feet). There are a number of scattered trees and bushes in close proximity to the alignment.

3.1.2 Reach 5

The Reach 5 alignment continues westward from the confluence of Reaches 4 and 14 through agricultural land and follows the existing Llagas Creek alignment. Between about Stations 516+00 and 518+00, it passes underneath two existing Caltrans (Bridge No. 37-331 R/L) bridges at Highway 101 that are each supported on three spread footings foundation (two abutments and one bent). Design bearing capacity is 239.4kPa (kilo Pascals) [5,000 pounds per square foot (psf)] for the abutments and 383.0kPa (8,000 psf) at the center bent. There are several scattered one-story houses and farm buildings set back at relatively moderate to large distances from the proposed channel. The ground surface drops in an easterly direction from about Elevation 252 to 245 Elevation.

3.1.3 Reach 6

The Reach 6 alignment follows the existing Llagas Creek alignment northward where it bypasses Lake Silveira. At the south end of the alignment, it parallels several deep water percolation ponds (that were formerly gravel pits); shallow standing water was observed in these ponds during field exploration. The south end is primarily agricultural with a number of scattered one-story houses and greenhouses set back from the proposed channel. There are a number of scattered trees and bushes in close proximity to the south end and middle of the alignment.

North of Church Avenue (between about Station 583+00 and 594+00), the west edge of Llagas Creek parallels the east shoulder of Llagas Avenue; the existing west channel face is concrete lined within this 1,100 foot-long segment. In this area and northward there is both urban development and agricultural use. In the vicinity of Station 606+00 to 615+00, there is a County water treatment plant and transfer station on the north side of the alignment. The existing and new alignment crosses underneath bridges at Church Avenue (about Station 552+00), East San Martin Avenue (about Station 630+00), Llagas Avenue (about Station 675+50), and underneath a railroad at about Station 690+00 and Monterey Road at about Station 692+00. North of Monterey Road up to Lake Silveira, the alignment passes primarily through undeveloped County of Santa Clara (County) property, which abuts agricultural land. The ground surface drops in a southerly direction from about Elevation 317 to 252.

Levee 6A is setback from and north of Llagas Creek primarily on County property between Monterey Road and Lake Silveira. The middle of Levee 6A abuts commercial/industrial property containing buildings, parked trucks and stored materials.

Levee 6B is setback from and northwest of Llagas Creek and Lake Silveira on County property; it abuts open agricultural land.

3.1.4 Reach 7

The portion of Reach 7A, which extends from Lake Silveira to Watsonville Road crosses relatively flat land that is primarily agricultural. From Watsonville Road northward to about Edmunson Avenue, the alignment of the remainder of Reach 7A follows the alignment of Llagas Creek; the adjacent Morgan Hill urban setting ranges from residential dwellings to commercial/retail buildings. Just north of Watsonville Road, some of these houses are in close proximity to the existing channel hinge point. The ground surface drops in a southerly direction from about Elevation 325 to 317.

The alignment of Reach 7B passes through Morgan Hill between Edmunson Avenue and West Dunne Avenue along the existing Llagas Creek alignment. The adjacent urban setting ranges from commercial/industrial to residential dwellings. In particular, the alignment between about Ciolino Avenue and West Dunne Avenue will be in close proximity to commercial buildings in the Morgan Hill Plaza. The ground surface drops in a southerly direction from about Elevation 340 to 325.

3.1.5 Reach 8

The Reach 8 alignment extends through a highly urbanized area of Morgan Hill along the existing alignment of Llagas Creek from West Dunne Avenue to a point on Hale Avenue located about 1,200 feet north of Wright Avenue. The south end traverses through downtown, including a portion along Monterey Road. Several existing box culverts are located within this reach, including West Dunne Avenue, West Fifth Street, Monterey Road, West Second Street, Warren Avenue, West Main Avenue and Wright Avenue. North of West Dunne Avenue (about Station 149+50), an existing commercial building is also in close proximity to the alignment; the west end of this building is above grade and supported on concrete columns extending below ground surface (bgs). In addition at about Station 165+00 off West Second Street, a one-story apartment building is positioned immediately south of the alignment. The west end of Reach 8 traverses along Hale Avenue, adjacent to Morgan Hill High School, a church and numerous residential dwellings. The existing channel just east of Hale Avenue (between West Main Avenue and Wright Avenue is partially lined with sacked concrete riprap. The ground surface drops in a southerly direction from about Elevation 349 to 340.

3.1.6 Reach 14

The alignment of Reach 14 follows the alignment of East Little Llagas Creek through primarily undeveloped agricultural land. However, at a few locations, it is near existing greenhouses. In addition several commercial buildings are offset in the vicinity of Station 70+00. Between about Stations 130+00 and 184+00, the alignment parallels Highway 101. The existing and new alignment crosses underneath bridges at Church Avenue (about Station 27+50), East San Martin Avenue (about Station 97+50) and Sycamore Avenue (about Station 125+00), as well as a box culvert at East Middle Avenue (about Station 171+50). Considerable erosion was observed in the existing channel at about Station 120+00. The ground surface drops in a southerly direction from about Elevation 303 to 245.

3.2 SUBSURFACE CONDITIONS

3.2.1 Field Exploration

A total of 143 exploratory borings were drilled along the proposed project alignment. This includes 129 primary borings and 14 shallower supplementary borings. The shallower borings were drilled adjacent to 14 of the primary borings to obtain supplementary soil samples for laboratory testing. In addition, a total of 23 piezometers were also installed adjacent to 23 of the primary borings. The exploration program is summarized in the table below.

Type	Number of Exploration Points	Exploration Point Assigned Number
Primary Boring	129	SB 4-1 through SB 4-18, SB 5-1 through SB 5-5, SB 6-1B, SB 6-1C, SB 6-1D, SB 6-1 through SB 6-14,SB 6-8A, SB 6-16 through SB 6-28, SB 6-21B, SB 6-22B, SB 7A-1 through SB 7A-4, SB 7A-4B, SB 7A-4C, SB 7A-5B, SB 7A-6B, SB 7A-11B, SB 7A-12B, SB 7B-1B, SB 7B-2B, SB 7B-3B, SB 7B-3C, SB 7B-4B, SB 7B-4C, SB 7B-4D, SB 7B-4E, SB 7B-4F, SB 7B-5B, SB 7B-6C, SB 7B-6B, SB 8A-1, SB 8A-2, SB 8A-1B, SB 8A-2B, SB 8A-2C, SB 8A-2D, SB 8A-3, SB 8A-3B, SB 8A-3C, SB 8A-4, SB 8A-4B, SB 8A-4C, SB 8A-4D, SB 8A-4F, SB 8A-5, SB 8A-5B, SB 8A-6, SB 8A-6B, SB 8A-7, SB 8A-7B, SB 8A-8, SB 8A-8B, SB 8A-9, SB 8A-9B, SB 14-2 through SB 14-10, SB 14-11 (east), SB 14-12 through SB 14-27 and SB 14-26B
Supplementary Boring	3	Rotary wash and Pitcher barrel samples: SB 7A-2R, SB 7A-4BR and SB 14-18R
Supplementary Boring	11	Hollow stem auger and additional samples: SB 14-7A, SB 14-10A, SB 14-11 east A, SB 14-11 west, SB 14-11 west A, SB 14-12A, SB 14-16A, SB 14-18A, SB 14-20A, SB 14-22A and SB 14-25A
Piezometer	23	OW 4-6, OW 4-11, OW 4-13, OW 4-18, OW 5-3, OW 6-7, OW 6-12, OW 6-17, OW 6-25, OW 6-27, OW 7A-2, OW 7B-4B, OW 8A-1B, OW 8A-2C, OW 8A-3B, OW 8A-4, OW 8A-5, OW 14-3, OW 14-10, OW 14-16, OW 14-22, OW 14-26 and OW 14-27

All borings were drilled in the period between September 20 and November 14, 2005. For most of this period, either one or two hollow stem auger rigs were used. However, a rotary wash rig was used during the period of October 6 through October 14, 2005 to drill 13 primary borings: SB 4-2, SB 4-3, SB 4-5, SB 4-6, SB 4-7, SB 4-8, SB 4-9, SB 4-11, SB 4-14, SB 4-15, SB 4-16, SB 4-17 and SB 4-18. In addition, the rotary wash method was also used to drill supplementary borings SB 7A-2, SB 7A-4B, and SB 14-18 on October 14, 2005 to obtain Pitcher barrel samples. Furthermore, on November 2, 2005 three primary borings SB 14-26B (located in the bottom of Llagas Creek), SB 6-8 (located in a confined fenced in area) and SB 6-8A (also in a fenced in area) were drilled using a portable hydraulic rotary drill rig. After Boring SB 6-8 could not be advanced deeper than 10 feet, this rig was offset several feet and Boring SB 6-8A was attempted; however Boring SB 6-8A was terminated at 2½ feet when refusal was met at a concrete slab.

All borings were drilled and piezometers installed under the supervision of a Professional Civil Engineer, Geotechnical Engineer, Geologist or Certified Engineering Geologist from URS and Pacific Geotechnical Engineering, including Pitipat Preedanont (P.E.), Anne-Marie Moore (P.E., G.E.), Terrence Carroll (P.E.), G. Reid Fisher (R.G., C.E.G.), Daniel Peluso (P.E., G.E.), Soma Goresky (C.E., G.E.) and Peter Anderson (P.G., C.E.G.).

Boring depths generally were about 40 feet (39.0 to 41.5 feet), except as follows:

Boring Number	Depth (feet)	Comment
SB 4-5	31½	Rotary wash cave-in at 31.5 feet in gravels
SB 4-6	36½	Rotary wash, sampler refusal at 36.5 feet
SB 4-9	35	Rotary wash, drilled to 35 feet but caved in at 30 feet.
SB 6-8	10	Hole collapsed below 6 feet using portable hydraulic rotary drill rig. Miscellaneous fill from 6 to 9 feet with wires, pvc pipe, plastic bags etc. Hole offset twice, third hole SB 6-8A met refusal at 2.5 feet with concrete slabs.
SB 7A-2R	8	Pitcher barrel sampling
SB 7A-4BR	8	Pitcher barrel sampling
SB 8A-4C	33½	Terminated 8 feet into bedrock
SB 8A-5B	29	Terminated 12 feet into bedrock
SB 14-7A	7½	Resample clay at 7 feet
SB 14-10A	6½	Resample clay at 3 and 6 feet
SB 14-11 (east)	25	Supplement SB 14-11 (west) to 40 feet.
SB 14-11 east A	7½	Resample clay at 4 and 7 feet
SB 14-11 west A	4	Resample clay at 4 feet
SB 14-12A	3½	Resample clay at 3 feet
SB 14-16A	20	Resample clay at 5, 10, 15 and 20 feet
SB 14-18R	9½	Pitcher barrel sampling
SB 14-18A	20	Resample clay at 5, 10, 15 and 20 feet
SB 14-20A	6½	Resample clay at 3 and 6 feet
SB 14-22A	9	Resample clay at 3, 6 and 9 feet
SB 14-25A	9½	Resample clay at 4 and 8 feet
SB 14-26B	20	Hole cave-in at 20.0 feet at bottom of creek using portable hydraulic rotary drilling.

All piezometers were installed to a depth of 6.1m (20 feet) below ground surface.

Most boreholes were advanced with a truck-mounted drill rig using 200mm (8-inch) diameter, continuous-flight, hollow-stem auger. Thirteen primary borings and 3 supplementary borings were advanced with the rotary wash method; the hole diameter was 127mm (5 inches). Three borings (SB 6-8, SB 6-8A and SB 14-26B) were drilled using a 89mm (3½-inch) solid auger (with the portable hydraulic rotary drill rig). Representative samples were obtained at depths of 0.3, 0.9, 1.5m (1, 3, 5 feet) and every subsequent 1.5m (5 feet) below existing ground surface. Samples were not retrieved from the piezometer borings. Representative samples were retrieved with several types of samplers. In fine-grained soils, we used both the Modified California sampler with 51mm (2-inch) internal diameter and 64mm (2½-inch) outside diameter and the California (Dames and Moore) sampler with 64mm (2½-inch) internal diameter and 76mm (3 inch) outside diameter to retrieve relatively undisturbed samples; the sampling technique was performed in accordance with ASTM D1586 "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils". In granular soils, we used the Standard Penetration Test (SPT) sampler (35mm or 1⅜-inch inside diameter and 51mm or 2-inch outside diameter) and the California (Dames and Moore) sampler with 64mm (2½-inch) internal diameter and 76mm (3 inch) outside diameter; this sampling technique was also performed in accordance with ASTM D1586 "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils". After the borehole was advanced to the desired depth, the sampler was lowered to the bottom of the borehole and then driven about 460mm (18 inches) into the soil, with the blow count being recorded for each of three 152mm (6-inch) increments of driving length. In fine-grained soils, thin-walled Shelby tubes (73mm or 2⅞-inch inside diameter and 76mm or 3-inch outside diameter) were pushed hydraulically into the soil to obtain undisturbed samples; (1) this sampling technique was also performed and (2) the sampler was also in accordance with ASTM D1587 "Standard Practice for Thin-Walled Tub Geotechnical Sampling of Soils". The hydraulic pressure required to push the tube is shown on the boring logs.

The Pitcher barrel was also used in conjunction with the 76mm (3-inch) Shelby tube sampler (tube) in several borings including SB 7A-2, SB 7A-4B, and SB 14-8. Drilling fluid was circulated downward through the barrel and flushed cuttings from the bottom of the hole using the rock-coring technique. When the tube encountered the bottom it was pushed upward with respect to the cutter barrel, whereupon circulation was diverted downward in the annular space between tube and barrel, beneath the rotating cutter barrel, and upward alongside the barrel. If the soil was stiff to hard, the spring was compressed until the cutting edge of the tube was forced above the level of the bottom of the cutter barrel. As the barrel rotates it cuts an annular ring leaving a cylinder of soil over which the tube sampler slides and protects the sample against further erosion by the circulating fluid. The barrel has an outside diameter of 104mm (4.1 inches) and length of 610mm (24 inches).

In addition, disturbed bulk (bag) samples were collected from drill cuttings at depths indicated on the logs of borings. These samples were primarily used for laboratory tests including R-value and laboratory compaction tests. CU tests were performed on a remolded specimen derived from one of these bulk samples.

The Modified California and California samplers housed brass liners that held the soil samples as the sampler was driven. After the sampler was withdrawn from the borehole, the soil samples contained in the brass liners were exposed, examined, and field classified. Subsequently, the brass liners were sealed to preserve the soil natural moisture. All samples were transported to the URS San Jose soil laboratory for further examination. Soil classification was made in accordance with the Unified Soil Classification System (USCS) based on field and laboratory observations and laboratory test results; the classification of soil samples was performed in accordance with ASTM D2487 "Standard Practice for Classification of Soil for Engineering Purposes (Unified Soil Classification System)". The graphical logs of borings are presented in Appendix B of Volume II. The nomenclature and soil designations used in the USCS are presented in Figure B-1. Figure B-2 presents the legends and symbols used in the boring logs. Logs of Borings were then prepared and contain a summary of laboratory test results.

A typical standpipe piezometer is shown in Appendix C of Volume II; this construction was approved by the District. In addition, the graphical logs of observation wells are presented in Appendix C of Volume II.

3.2.2 Photographs

URS took photographs at boring locations during the field exploration program; these photographs are included electronically on the enclosed CD in Appendix B of Volume II.

3.2.3 Conditions Encountered during Field Exploration

Weather conditions during field exploration in September, October and November 2005, were generally sunny, clear and warm to hot (in the 60's, 70's or low 80's). In some cases, it was foggy in the morning, but either cleared up or became hazy in the afternoon. Exceptions to the above include the following:

- September 26, cloudy with a few sprinkles of rain
- October 12, cold and foggy
- November 2, cold and windy

In general, no rain or storms occurred during field exploration. In summary, it is our opinion that weather was mild and had no detrimental effect on drilling or sampling procedures.

Some surface conditions noted during drilling include the following:

- SB 4-8; boring is located 9.1m (30 feet) north of Llagas Creek, creek is dry to bottom (approximately 3.0m or 10 feet below ground surface)
- SB 4-9; boring is located 15.2m (50 feet) south of Llagas Creek, creek is dry to bottom (approximately 3.0m or 10 feet below ground surface)
- SB 4-14; boring is located 4.6m (15 feet) south of Llagas Creek, creek is dry to bottom (approximately 3.7m to 4.6m or 12 to 15 feet below ground surface)
- SB 4-15; boring is located 4.6m (15 feet) northwest of Llagas Creek, creek is dry to bottom (approximately 4.6m to 6.1m or 15 to 20 feet below ground surface)
- SB 4-16; boring is located 4.6m (15 feet) north of Llagas Creek, creek is dry to bottom (approximately 3.7m to 4.6m or 12 to 15 feet below ground surface)
- SB 4-18; boring is located 6.1m (20 feet) south of Llagas Creek, creek is dry to bottom (approximately 3.7m or 12 feet below ground surface)

- SB 7B-2B; boring is located 30.5m (100 feet) south of Llagas Creek, water in channel about 2.7m (9 feet) below top of bank
- SB 14-2; scattered cobbles to 102mm (4 inches), boulders on channel banks to 0.9m (3 feet) plus

3.2.4 Laboratory Testing

Soil samples were carefully sealed in the field and returned to our San Jose laboratory for visual observation. Based on our review of the soil samples in the San Jose laboratory, we then assigned laboratory tests and obtained the concurrence of the District. Subsequently, the soil samples were delivered to the URS Signet laboratory in Hayward for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing. Laboratory tests were performed on selected soil samples. These tests include, but are not limited to, water content, dry density and unconfined compressive strength. The results of these tests are presented at the corresponding sample locations on the Logs of Borings. Additional tests including Atterberg limits, sieve analysis, consolidation, and triaxial compression were made to better characterize the soil conditions and to estimate the compressibility and shear strength of the levee fill, foundation material and channel material.

The moisture content, dry density, unconfined compression strength and Atterberg limits (plasticity index liquid limit and plastic limit) are presented on the logs of borings in Appendix B of Volume II.

Furthermore, laboratory test results on native soils are presented in Appendix D of Volume III (Sections 1 through 8) as summarized below.

Appendix D of Volume III Section Number	Name of Laboratory Test	ASTM Test Number
1	Atterberg Limits (liquid limit, plastic limit and plasticity index)	D4318
2	Grain size distribution curves	D422
3	Laboratory Compaction	D1557
4	Unconfined Compression	D2166
5	Unconsolidated Undrained Triaxial Compression (UU)	D2850
6	Consolidated Undrained Triaxial Compression (CU) with pore pressure measurements	D4767
7	Consolidation	D2435
8	R-value	D2844

A more comprehensive discussion of the laboratory testing program is presented in Appendix D of Volume III.

The results of laboratory compaction tests are also presented in Figure 34.

To estimate the engineering properties of potential levee fill materials, laboratory tests were performed on a bulk sample of potential borrow material obtained from Boring SB 6-16 (depth 0.6m to 1.8m or 2 to 6 feet bgs). Laboratory test results on this clayey sand sample are presented in Appendix E of Volume III including laboratory density, unconfined compression and consolidated undrained triaxial compression with pore pressure measurements. The shear strength test samples were recompacted at about 2 percent wet of optimum moisture content (7.5 percent) and 92 percent relative compaction (on dry weight basis with maximum dry density of 21.6kN per cubic meter) 137.8 pcf.

3.2.5 Soil and Groundwater Conditions

We have based our assessment of the soil conditions primarily on the field exploration conducted for this study supplemented with the District's field exploration in 2003 (Reach 7B Box Culverts) and 2004 (Reach 7A). For convenience of discussion, the subsurface soil and groundwater conditions along the alignment are presented below with reference to the Corps hydraulic stations as shown on Figures 4 through 28. Furthermore 9 fence-diagrams presenting the log of borings in terms of stick logs and simplified soil classification symbols were developed for the seven reaches of the creek; the diagrams are presented as Figures 35 through 48. Groundwater measurements include those in the 2003 and 2004 District boring logs, the URS 2005 boring logs and the 2003, 2004, 2005 District piezometer readings and the 2005, 2006 readings at piezometers installed by URS. The piezometer readings are converted to elevations and listed in Table 16. More specifically, this includes for each Reach the date and groundwater elevations in tabular form and graphical form: Reach 4 (Table 16A), Reach 5 (Table 16B), Reach 6 (Table 16C), Reach 7A and 7B (Table 16D), Reach 8 (Table 16E) and Reach 14 (Table 16F).

3.2.5.1 Reach 4

Stations 375+22 to 396+00 (Borings SB 4-18 to SB 4-15)

This portion of Reach 4 can be generalized as a transition zone from a mostly granular depositional environment to more fine-grained soils such as clays and silts. The surficial deposits range from 2 to 10 feet bgs and are sandy, silty clay; the clays are underlain by clayey gravel and poorly graded sands. The granular deposits extend to 22 feet below ground surface (bgs) to the terminal depths of about 40 feet bgs. Below the sand and gravel layer, the clay layer becomes thicker towards the southern end of the reach (Station 375+22). The sand and gravel deposits are dense to very dense, whereas the silty clay and clay layers are medium stiff to stiff in consistency.

Groundwater levels were not measured in Borings SB 4-15 to SB 4-18 because the rotary wash method, used to drill these locations, is not reliable in revealing groundwater level measurements. However, these borings were dry to depths ranging from 3 to 8 feet bgs. Piezometer OW 4-18 was installed in this area and was dry (to a depth of 20 feet) during November and December, 2005; subsequently, groundwater was measured with the shallowest depth at 10.8 feet (Elevation 214.0) on January 6, 2006.

Stations 396+00 to 500+00 (Borings SB 4-14 to SB 4-1)

Granular deposits of clayey gravel, clayey sand, poorly graded sand, poorly graded gravel, well graded gravel and silty sand were primarily encountered from the ground surface to the terminal depths of borings in this segment. However, at a few locations including SB 4-2, SB 4-3, SB 4-5 and SB 4-11 a surficial layer of hard sandy lean clay was observed to depths ranging from 2½ to 8 feet bgs; at SB 4-4 a medium sandy silt layer extends to a depth of 4½ feet bgs. Occasional layers of 3 to 5 feet of sandy lean clay were also encountered at depths ranging from 15 to 26 feet. The sand and gravel deposits are dense to very dense, whereas the clay layers are stiff to hard in consistency.

Measured groundwater depths at the time of drilling in hollow stem auger borings ranged from 11½ feet (Elevation 233.2 in Boring SB 4-1) to 39½ feet bgs (Elevation 184.4 in Boring SB 4-13). Piezometers OW 4-6, OW 4-11 and OW 4-13 were installed in this area. Shallowest groundwater depths during 2005 and 2006 at these three piezometers were 9.1 feet (Elevation 232.6), 8.2 feet (Elevation 224.6) and 5.8 feet (Elevation 221.4), respectively on January 6, 2006. In summary groundwater levels encountered in the borings and piezometers just described showed variations between about Elevations 184 and 233 feet, with an apparent down gradient from north to south.

3.2.5.2 Reach 5

Stations 501+00 to 525+50 (Borings SB 5-5 to SB 5-1)

At only one Boring SB 5-1, a surficial layer of very stiff sandy lean clay was encountered to a depth of 4 feet bgs. In general, granular deposits of silty sand, clayey sand and poorly graded sand extend to terminal depths (approximate Elevation 206 to 208) of exploratory borings within this reach. This deposit grades coarser to clayey gravel towards the northeasterly direction near Boring SB 5-4. A layer of sandy lean clay was encountered between depths of 8 and 17 feet (Elevation 232 and 242) in Boring SB 5-4 and extends within Boring SB 5-3 and SB 5-5, with an average thickness of 4 feet, between about Stations 501+00 to 514+00. The granular deposits are dense to very dense, whereas the clay layer was stiff to very stiff.

Depth to groundwater ranged from 9½ feet (Elevation 235.6 in Boring SB 5-2) to 20½ feet bgs (Elevation 228.2 feet in Boring SB 5-4). Piezometers OW 5-3 and OW 14-27 were installed in this area; the shallowest groundwater depth measured was 6.3 and 9.1 feet (Elevation 243.4 and 241.2) respectively, on January 6, 2006. Groundwater levels encountered in the borings and piezometers just described showed variations between about Elevations 228 and 243 feet, with an apparent down gradient from a southwesterly to northeasterly direction.

3.2.5.3 Reach 6

Station 525+50 to 576+00 (Borings SB 6-28 to SB 6-22)

Surficial fill was encountered in Borings SB 6-22, SB 6-25 and SB 6-27 to depths ranging from 4½ to 5½ feet bgs. Fill materials range from granular (medium dense to very dense clayey gravel, silty gravel, clayey sand and silty sand) to stiff sandy lean clay.

Native granular deposits of silty sand, poorly graded sand, well graded sand and clayey sand, grading to silty gravel, poorly graded gravel and clayey gravel, were revealed to terminal depths of borings within this segment of Reach 6. Discontinuous layers of lean clays and sandy lean clays were encountered between approximately Elevation 232 and 240 or about 18½ to 29 feet bgs. The sand and gravel deposits are dense to very dense.

Depth to groundwater in borings ranged from 7 feet (Elevation 254.2 in Boring SB 6-23) to 16 feet (Elevation 249.5 in Boring SB 6-22) bgs. Piezometer OW 6-25 was installed in this area; the shallowest groundwater depth was 12.6 feet (Elevation 254.3) on January 6, 2006. Groundwater levels encountered in the borings and piezometer just described showed variations between approximately Elevations 249 and 254 feet, with an apparent down gradient trend in the southerly direction.

Station 576+00 to 605+00 (Borings SB 6-21B to SB 6-18)

In general, medium dense to very dense clayey sand, well graded sand, poorly graded sand, poorly graded gravel, clayey gravel and well graded gravel overlie deposits of stiff to very stiff sandy lean to fat clays. Approximately 24 to 32 feet of these sand and gravel deposits extend to about Elevation 236 to 248. In addition a surficial stiff silt layer with sand was encountered to a depth of 2½ feet bgs in Boring SB 6-18.

Groundwater depths encountered in two borings ranged from 9 feet (Elevation 266.6 in Boring SB 6-18) to 12 feet bgs (Elevation 261.3 in Boring SB 6-20). One piezometer OW 6-17 was installed in this area; the shallowest groundwater depth was 6.1 feet (Elevation 273.6) on January 6, 2006. In summary, the borings and piezometer just described show groundwater variations between approximately Elevations 261 and 274 feet, with an apparent down gradient trend in the southerly direction.

Station 605+00 to 692+00 (Borings SB 6-17 to SB 6-5)

Surficial fill was encountered in a number of Borings including SB 6-6, SB 6-7, SB 6-8, SB 6-8A, SB 6-10, SB 6-11, SB 6-12, SB 6-13 and SB 6-14 to depths of 2½ to 8½ feet bgs. The types of fill ranged considerably from loose to medium dense poorly graded gravel, loose silty sand, loose clayey gravel, loose to very dense poorly graded sand, medium dense clayey sand and medium sandy lean clay. Due to limited access at Borings SB 6-8 and SB 6-8A, a portable drill rig was used; fill was encountered in both borings. At Boring SB 6-8A refusal was met at 2½ feet bgs when a concrete slab was encountered.

Underlying the fill (at the above listed borings), or beginning at the ground surface in the remainder of other borings, primarily granular deposits were encountered. These include poorly graded gravel, well graded gravel and clayey gravel, interbedded with gradational changes to clayey sand, poorly graded sand and well graded sand generally in the upper 23½ to 39 feet (approximate Elevation 254 to 256 feet) of the borings within this segment. Below the dense to very dense granular deposits, medium stiff to stiff lean clays sometimes extend to the terminal depths of the exploratory borings (approximate Elevation 242 to 252 feet). However at Boring SB 6-5, a medium lean clay interbed was encountered between depths of 8½ to 13½ feet bgs.

Depth to groundwater in borings ranged from 6 feet (Elevation 270.9 in Boring SB 6-17) to 34½ feet bgs (Elevation 249.9 in Boring SB 6-13); Borings SB 6-8 and SB 6-8A were dry to the terminal depths of 10 and 2½ feet bgs, respectively. Three piezometers were installed in this area including OW 6-7, OW 6-12 and OW 6-17; shallowest groundwater depths during 2005 and 2006 were 9.6 feet (Elevation 292.6), 7.1 feet (Elevation 282.5) and 6.1 feet (Elevation 273.6), respectively on January 6, 2006. Groundwater levels encountered in the borings and piezometers just described showed variations between Elevations 250 and 293 feet, with an apparent down gradient trend in the westerly direction.

Levee 6A Station 692+00 to 718+00 (Borings SB 6-4 to SB 6-1)

Fill, consisting of medium dense clayey sand, was encountered in Boring SB 6-1 to a depth of 5 feet bgs.

The predominate native soil conditions along Levee 6A are granular, including well graded to poorly graded to clayey gravel and poorly graded to well graded to silty to clayey sand which extend to the terminal depth. These granular layers range in relative density from dense to very dense and occasionally medium dense.

Depth to groundwater in the borings ranged from 12 feet (Elevation 286.4 in Boring SB 6-10) to 27½ feet bgs (Elevation 279.4 in Boring SB 6-3). No piezometers were installed in this area. In summary, the borings just described show groundwater levels varying from about Elevation 279 to 286.

Levee 6B (Borings SB 6-1B, SB 6-1C and SB 6-1D)

With the exception of a 4½ foot thick surficial layer of medium clay in SB 6-1C, the native soils are predominately granular: medium dense to very dense silty sand, clayey sand, poorly graded sand and clayey gravel, well graded gravel and poorly graded gravel. A layer of medium to stiff lean clay was encountered in SB 6-1B from a depth of 37 to 40 feet bgs.

Groundwater depths in these borings ranged from 12 feet (Elevation 303± in Boring SB 6-1D, assuming ground surface at SB 6-1D is about Elevation 315) to 14 feet bgs (Elevation 299.7 in Boring SB 6-1C) to 23½ feet bgs (Elevation 287.3 in Boring SB 6-1B). No piezometers were installed in this area. The borings just described indicate groundwater ranges from about Elevation 287 to 303. Based on a drawing received from the District on March 2, 2006, the surveyed water level in nearby Lake Silveira is Elevation 303.8 feet, which corresponds closely to the estimated water elevation in Boring SB 6B 6-1D.

3.2.5.4 Reach 7A

Stations 11+00 to 100+00 (Borings SB 7A-13 exist to SB 7A-1)

In Reach 7A we reviewed nine (9) previous 2004 logs of Borings SB 7A-5 exist through SB 7A-13 exist, as well as ten (10) 2005 Borings including SB 7A-1, SB 7A-2, SB 7A-3, SB 7A-4, SB 7A-4B, SB 7A-4C, SB 7A-5B, SB 7A-6B, SB 7A-11B and SB 7A-12B.

Within Reach 7A, granular layers were encountered beginning at the ground surface in Borings SB 7A-13 exist, SB 7A-12 exist, SB 7A-10 exist, SB 7A-9 exist and SB 7A-1; these layers include mostly medium dense to very dense silty sand, clayey sand, well graded sand, well graded gravel and poorly graded gravel and extend to depths of about 4½ to more than 30 feet bgs (terminal depth). Occasional interbeds of very stiff to hard silty clay and lean clay were encountered in these five borings.

In the remainder of the borings in Reach 7A, fine-grained soils were encountered from the ground surface to depths ranging from 3 to 17½ feet bgs; this included hard lean clay fill in Boring SB 7A-2 (to 5 foot depth), whereas native surficial clay in other borings was very stiff to hard lean clay and sandy lean clay, as well as stiff fat clay. Underlying these fine grained deposits are mostly dense to very dense granular layers (clayey gravel, well graded gravel, clayey sand, well graded sand and poorly graded sand) to the terminal depth of borings (30 to 40 feet bgs), with thin to thick very stiff lean clay interbeds. One exception is Boring SB 7A-2 where very stiff to hard lean clay extended from 11½ to 40 feet bgs.

Depth to groundwater in the 2004 and 2005 borings ranged from 15 feet (Elevation 297.2 in Boring SB 7A-12B and Elevation 299.8 in Boring SB 7A-12 exist) to 32 feet bgs (Elevation 293.1 in Boring SB 7A-1). Piezometer OW 7A-2 was installed in this area; the shallowest water depth in 2005 and 2006 was 13.6 feet bgs (Elevation 316.4) on March 14, 2006.

Four piezometers were installed in June 2004 by the District including P 7A-9 exist, P 7A-10 exist, P 7A-11 exist and P 7A-H4 exist; water depths were measured at these piezometers in 2004 and January and March 2005. The shallowest depths to groundwater in 2004 and 2005 were 26.5 feet (Elevation 297.0 in P 7A-9 exist on June 9, 2004), 9.7 feet (Elevation 310.8 in P 7A-10 exist on March 9, 2005), 9.7 feet (Elevation 309.7 in P 7A-11 exist on March 9, 2005) and 8.0 feet (Elevation 307.0 in P 7A-H4 exist on March 9, 2005).

In summary, groundwater levels in the borings and piezometers described above range from approximately Elevation 293 to 316.

3.2.5.5 Reach 7B

Station 100+00 to 148+00 (Borings SB 7B-6C to SB 7B-1B and SB 7B-1 exist to SB 7B-7 exist)

In Reach 7B we reviewed seven (7) previous 2003 logs of Borings SB 7B-1 exist through SB 7B-7 exist, as well as twelve (12) 2005 Borings including SB 7B-1B, SB 7B-2B, SB 7B-3B, SB 7B-3C, SB 7B-4B, SB 7B-4C, SB 7B-4D, SB 7B-4E, SB 7B-4F, SB 7B-5B, SB 7B-6B and SB 7B-6C.

Within Reach 7B subsurface conditions are variable and it is difficult to generalize. Nevertheless, most borings reveal a surficial layer of cohesive soil to depths of 2 to 8½ feet bgs underlain by alternating layers of granular soils and cohesive soils to the terminal depth (40 to 48 feet bgs).

Medium sandy lean clay fill was encountered in Boring SB 7B-4F to a depth of 5 feet bgs, whereas dense silty sand fill was found in SB 7B-7 exist to a depth of 3½ feet bgs. The other native surficial cohesive deposits are very stiff to hard sandy lean clay, lean clay, fat clay and sandy silt. Surficial granular soils consisting of medium dense clayey sand were encountered to 2 feet to 4½ feet bgs in Borings SB 7B-6B and SB 7B-6C, respectively. Structural pavement sections were encountered in SB 7B-1 exist, SB 7B-3 exist, SB 7B-4 exist, SB 7B-5 exist and SB 7B-6 exist.

The deeper granular layers include mostly dense to very dense clayey gravel, well graded gravel, poorly graded gravel, poorly graded sand, clayey sand, well graded sand and silty sand. Most of the alternating cohesive soils are stiff to hard (occasionally medium) sandy lean clay, lean clay, fat clay and sandy silt.

Groundwater depths in the 2003 and 2005 borings ranged from 6½ feet (Elevation 331.7 in Boring SB 7B-1) to 39½ feet bgs (Elevation 284.5 in Boring SB 7B-6B); Boring SB 7B-4F was dry to 40 feet at time of drilling. Nearby piezometer OW 7B-4B revealed the shallowest water depth in 2005 and 2006 was 3.7 feet bgs (Elevation 328.5) on March 14, 2006.

Four piezometers were installed in June 2003 by the District including P 7B-2 exist, P 7B-3 exist, P 7B-5 exist and P 7B-7 exist; water depths were measured at these piezometers in 2003, 2004 and January and March 2005. The shallowest depths to groundwater in 2003, 2004 and 2005 were 5.6 feet (Elevation 333.0 in P 7B-2 exist on March 9, 2005), 4.6 feet (Elevation 334.4 in P 7B-3 exist on March 9, 2005), 5.9 feet (Elevation 333.1 in P 7B-5 exist on March 9, 2005) and 5.5 feet (Elevation 333.5 in P 7B-7 exist on March 9, 2005).

In summary, groundwater levels in the borings and piezometers described above range from approximately Elevation 285 to 334.

3.2.5.6 Reach 8A

Stations 148+00 to 170+00 – South Segment (Borings SB 8A-1 to SB 8A-4F)

The South Segment generally includes an intermittent shallow layer of lean clay overlying predominately granular deposits and extends from about Station 148+00 to Warren Avenue (about Station 170+00) and includes Borings SB 8A-1, SB 8A-1B, SB 8A-2, SB 8A-2B, SB 8A-2C, SB 8A-2D, SB 8A-3, SB 8A-3B, SB 8A-3C, SB 8A-4, SB 8A-4B, SB 8A-4C, SB 8A-4D and SB 8A-4F. Fill, consisting of loose to very dense clayey sand and medium to stiff sandy lean clay to hard fat clay, was encountered to depths of 2½ to 8 feet bgs in Borings SB 8A-1, SB 8A-1B, SB 8A-2B, SB 8A-2C, SB 8A-2D, SB 8A-3B, SB 8A-3C, SB 8A-4B, SB 8A-4C and SB 8A-4F; a pavement section was also noted in Borings SB 8A-1B, SB 8A-2, SB 8A-2B, SB 8A-2C, SB 8A-2D, SB 8A-3B, SB 8A-4B, SB 8A-4C and SB 8A-4D. A surficial layer of hard sandy lean clay was noted in Borings SB 8A-3 and SB 8A-4 to depths of 2½ to 5½ feet bgs. This lean to fat clay layer was also noted underlying some of the fill to depths ranging from 5 to 8 feet bgs.

Within the 14 borings of the South Segment, native granular deposits predominately occurred including medium dense to very dense clayey gravel, poorly graded gravel, clayey sand and poorly graded sand to depths ranging from 22½ feet to 40 feet bgs; relatively impermeable greenstone was encountered beginning at depths of 17½ to 38 feet bgs in Borings SB 8A-3, SB 8A-3B, SB 8A-3C, SB 8A-4B, SB 8A-4C, SB 8A-4D and SB 8A-4F. In general, some stiff to hard lean clay interbeds were also encountered.

Depths to groundwater in the 2005 borings ranged from 18 feet (Elevation 322.5 in Boring SB 8A-1) to 34 feet bgs (Elevation 307.0 in Boring SB 8A-2B); Borings SB 8A-3, SB 8A-3B, SB 8A-3C, SB 8A-4C, SB 8A-4D and SB 8A-4F were dry at time of drilling to terminal depths of 30 to 40 feet bgs. Five piezometers OW 8A-1B, OW 8A-2C, OW 8A-3B, OW 8A-4 and OW 8A-5 were installed along this alignment. The shallowest groundwater depths in these piezometers, respectively, were 6.8 feet (Elevation 334.1), 7.9 feet (Elevation 334.9), 8.2 feet (Elevation 336.1), 6.6 feet (Elevation 338.7) and 9.0 feet (Elevation 338.2) bgs taken on March 14, 2006 except for OW 8A-2C taken on January 20, 2006.

In summary, groundwater levels in the borings and piezometers described above range from approximately Elevation 307 to 339.

Stations 170+00 to 206+00 – Modified North Segment (Borings SB 8A-5 to SB 8A-9B)

The Modified North Segment extends from Warren Avenue (about Station 170+00) to about Station 206+00 and includes Borings SB 8A-5, SB 8A-5B, SB 8A-6B, SB 8A-7, SB 8A-7B, SB 8A-8, SB 8A-8B, SB 8A-9 and SB 8A-9B; this segment consists primarily of lean clays and fat clays and occasional elastic silts. Fill, consisting of stiff to very stiff lean clay was encountered to a depth of 3 feet bgs in Boring SB 8A-7. A pavement section was encountered in Borings SB 8A-6B, SB 8A-7, SB 8A-7B, SB 8A-8, SB 8A-8B and SB 8A-9.

Underlying the fill, or beginning at the ground surface in the reminder of the other borings, primarily cohesive deposits were encountered. These include mostly stiff to hard lean clays, sandy lean clays, fat clays and sandy elastic silts which extend to the terminal depth of most borings. However, relatively impermeable greenstone was encountered beginning at a depth of 17 feet bgs in Boring SB 8A-5B. Occasional granular interbeds were encountered in several Borings including SB 8A-5, SB 8A-6, SB 8A-6B, SB 8A-7B and SB 8A-9; these interbeds include medium dense to dense poorly graded sand and clayey sand.

Depth to groundwater in borings ranged from 17 feet (Elevation 329.1 in Boring SB 8A-6B) to 39 feet bgs (Elevation 310.0 in Boring SB 8A-8B); Boring SB 8A-5B was dry at time of drilling to terminal depth of 29 feet bgs. One piezometer OW 8A-5 was installed in this area; the shallowest groundwater depth was 9.0 feet bgs (Elevation 338.2) on March 14, 2006.

As part of another study, the District has been periodically monitoring water depths in a groundwater monitoring well (09S03E20K003) located south of the intersection of Hale Avenue and Hillwood Lane. This groundwater monitoring well is also positioned about 400 feet northwest of Boring SB 8A-9B. Groundwater depth readings were taken at about 1-month intervals during the period of December 28, 2004 through April 11, 2006; deepest groundwater and shallowest groundwater also occurred on those same dates at depths of 29.7 feet (Elevation 322.7) and 8.1 feet (Elevation 344.3), respectively. This shallowest depth of 8.1 feet is similar to the shallowest depth of 9.0 feet measured in OW 8A-5, even through OW 8A-5 is at considerable distance from the groundwater monitoring well.

In summary, groundwater levels in the borings and piezometers described above range from about Elevation 310 to 338.

3.2.5.7 Reach 14

Stations 0+00 to 130+00 (Borings SB 14-27 to 14-9)

This area includes an intermittent shallow layer of lean clay and fat clay overlying predominately granular deposits. However, at a few locations the deeper granular deposits are minimal or are non-existent such as at Borings SB 14-18 and SB 14-16.

The surficial layers of stiff to hard lean clay, sandy lean clay and fat clay were encountered in all but five (Borings SB 14-13, SB 14-14, SB 14-17, SB 14-19 and SB 14-26B in channel bottom) of the 19 borings. These cohesive deposits extended to depths ranging from 3 to 22 feet bgs. In addition, fill was encountered in Borings SB 14-12, SB 14-15, SB 14-16A and SB 14-17 to depths ranging from ½ foot to 1½ feet bgs, consisting of loose clayey sand and clayey gravel.

Underlying the fill and surficial cohesive layers (and beginning at ground surface in Borings SB 14-13, SB 14-14, SB 14-17, SB 14-19 and SB 14-26B) are numerous granular layers to the terminal depth of most borings. The granular layers consist of medium dense to very dense clayey gravel, well graded gravel, clayey sand, poorly graded sand and silty sand. However, in Boring SB 14-7 this clayey sand layer was loose to a depth of 6½ feet bgs. Within the granular deposits are thin to thick interbeds of cohesive deposits ranging from stiff to very stiff silt, medium to hard lean clay and silty clay, to hard fat clay. At Borings SB 14-16 and SB 14-18 most of the soil profile consists of these cohesive deposits.

Depth to groundwater in borings ranged from 17½ feet (Elevation 269.1 in Boring SB 14-9 and Elevation 243.4 in Boring SB 14-21) to 34 feet bgs (Elevation 236.0 in Boring SB 14-16). Five piezometers were installed in this area including OW 14-10, OW 14-16, OW 14-22, OW 14-26 and OW 14-27; shallowest groundwater depths during 2005 and 2006 were 7.0 feet (Elevation 284.0), 4.7 feet (Elevation 268.3), 10.0 feet (Elevation 253.6), 9.0 feet (Elevation 243.4) and 9.1 feet (Elevation 241.2), respectively, taken on March 14, 2006 except January 6, 2006 taken at OW 14-27.

In summary, groundwater levels in the borings and piezometers described above range from approximately Elevation 236 to 284.

Stations 130+00 to 183+00 (Borings SB 14-8 to 14-2)

Fill, consisting of loose to medium dense clayey sand and very stiff lean clay, was encountered in Borings SB 14-2, SB 14-5, SB 14-7 and SB 14-8 to depths of 3½ to 5½ feet bgs.

The predominate native soil conditions are granular, including poorly graded to clayey gravel and poorly graded to silty to clayey sand that extend to the terminal depth. These granular layers range in relative density from dense to very dense and occasionally medium dense. In general, cohesive interbeds were also encountered, ranging from stiff to very stiff lean clay and sandy lean clay to very stiff silt.

Depth to groundwater in the borings ranged from 14½ feet (Elevation 279.5 in Boring SB 14-7) to 20½ feet bgs (Elevation 283.7 in Boring SB 14-3). One piezometer was installed in this area, OW 14-3; shallowest groundwater depth during 2005 and 2006 was 9.1 feet (Elevation 298.3) on March 14, 2006.

In summary, groundwater levels in the borings and piezometers described above range from approximately Elevation 279 to 298.

3.2.5.8 Groundwater Considerations

Water level readings taken during and immediately after completion of the boring operations are noted on the boring logs. In addition, groundwater level readings taken at the piezometer locations are included in Table 16. Cohesive soils were encountered in some of the borings, particularly in Segments 7B and 8; in cohesive soils a fairly long time would be required for the groundwater to seep into the bore hole and attain an equilibrium position with the long-term hydrostatic groundwater table. Thus, the immediate readings obtained may or may not be representative of the actual groundwater table level. For that reason, piezometers were installed and long-term water levels measured, particularly where cohesive soils were encountered. Furthermore, seasonal fluctuations in the location of the long-term hydrostatic groundwater table should be anticipated throughout the year depending upon variations in precipitation, evaporation, surface runoff and the water levels in Upper Llagas Creek and the adjacent Lake Silveira.

3.2.6 Summary of Field Corrected Blow Counts and Selected Laboratory Tests for Each Reach

Based on the results of the field exploration and laboratory tests performed (as described earlier in this section), we have prepared figures (on a reach by reach basis) of effective overburden vs. corrected blow counts (SPT), corrected blow counts vs. depth bgs, moisture content vs. depth bgs, dry density vs. depth bgs and unconfined compressive strength vs. depth bgs in the following order:

- Reach 4 – Figures 49 through 53
- Reach 5 – Figures 54 through 58
- Reach 6 – Figures 59 through 64
- Reach 7A and 7B – Figures 65 through 70
- Reach 8 – Figures 71 through 76
- Reach 14 – Figures 77 through 82

The correction factors used to estimate the equivalent SPT blow counts are summarized as follows:

Sampler Description	Correction Factor (CF) for Blows Per Foot (N)
2-inch O.D. and 1⅜-inch I.D. Standard Split Spoon sampler (SPT)	1.0
2½-inch O.D. and 2-inch I.D. Modified California sampler	For N≤ 20, CF=1.0 For N> 20, CF=0.8
3-inch O.D. and 2½-inch I.D. California (Dames and Moore) sampler	0.65

Except for Figures 53, 58, 64, 70, 76 and 82, it should be noted that the data presented in Figures 49 through 82 include coarse-grained and fine-grained soils. Since unconfined compressive strength vs. depth is shown in Figures 53, 58, 64, 70, 76 and 82, the test results are limited exclusively to fine-grained (lean clay and fat clay) soils.

From the CU triaxial compression test results exclusively on native clays (lean clays and fat clays), a plot of p' vs. q' is presented on Figure 83. Please note that the lower half includes results for all six reaches, whereas the upper half includes results for p' less than 3,500 pounds per square foot (psf). A straight line equation was estimated for both the upper half and the lower half of Figure 83, using the method of least squares. The equations are indeed linear and the scatter of data is minimal.

SECTION 4

4.1 CROSS SECTIONS SELECTED FOR ANALYSIS

4.1.1 Selection of Cross Sections

After detailed review and discussion with the District, seventeen (17) critical cross sections were mutually selected for slope stability analyses. The approximate locations of the 17 cross sections are shown in plan on Figure 3. A summary of the selected cross sections is presented in the following table.

Reach Number	Selected Cross Sections					
	Section Number	Approximate Corps Hydraulic Station*	Approximate Corps Design Station**	Nearest Boring(s)	Comment	Soil Type in Upper 10 ft to 15 ft
4	1	378+00	378+00	SB 4-18	Cut 14 ft	SM, SP, SC
4	2	472+00	472+00	SB 4-5	Cut 15 ft	3' CL, GM, SP, SW
5	3	518+00	518+00	SB 5-2	Cut 17.5 ft	SP, SC
6	4	573+00	573+00	SB 6-22	Cut 20.5 ft	GC, CL, GM, SC
6	5	619+00	619+00	SB 6-16	Cut 12 ft (right) Fill 6 ft (left)	5' ML, SW
6	6	673+00	673+00	SB 6-7	Partial cut 14 ft Total slope height 20 ft	SP-SM
6 (Levee 6A)	7	718+00	23+20 (Levee Station)	SB 6-1 SB 6-1B	New levee 5.5 ft high adjacent to channel 10 ft high	SM, GW-GC
6 (Levee 6B)	8	Not available	7+50 (Levee Station)	SB 6-1C SB 6-1D	New levee 5 ft high adjacent to channel 9 ft high	5' CL-ML, SP
7A (Berm)	9	73+60	62+00	SB 7A-5 Exist SB 7A-4C	Cut 3 ft, berm 1 ft, near houses, no cross section sheets readily available	8' CL, SC, GC
7A (Berm)	10	26+00	14+00	SB 7A-11B	Cut 10 ft; berm 1 ft	2' CL, 3' CH, SC
7B (Berm)	11	102+50	90+00	SB 7B-6B	Cut 10 ft; berm 4 ft	2' SC, 2' CL, SC, GC
7B (Berm)	12	123+60	111+60	SB 7B-3C	Cut 10 ft, berm 2 ft, adjacent to building structure (Corps)	3' CL, SC, GC, SW
8	13	151+00	151+00	SB 8A-1 SB 8A-1B	Cut 10 ft, gabions	SC, GC
8	14	165+00	165+00	SB 8A-4 SB 8A-4B	Cut 9 ft, gabions	CL, SC, CL
14	15	9+00	9+00	SB 14-26 SB 14-26B	Cut 17.5 ft	5' CL, SP-SM, CL, GC
14	16	170+00	170+00	SB 14-3	Cut 12 ft, total slope height 15 ft	SC
7A	10A	46+00	34+00	SB 7A-9 Exist	Cut 15 ft	10' SM, GW-GM

* The "Corps Hydraulic Stationing" is used in the Corps Sediment Assessment and Hydraulic Design Office Report including the Appendices, the Bestor survey information, and the District's Soil Boring Site Plan.

** The "Corps Design Stationing" is used in the Corps Preliminary design plans (plan and profile sheets) and the Corps Design Cross Sections.

The location of Levee 6A in Reach 6 (where Section Number 7 is positioned) is shown on Figures 14 and 15, whereas the location of Levee 6B in Reach 6 (where Section Number 8 is positioned) is shown on Figure 15.

In general, the Corps design of cross sections is based on (1) cut on one side of the channel and (2) leaving the other side of the channel unchanged.

4.1.2 Selection of Shear Strength Parameters

4.1.2.1 Introduction

In addition to the data presented in Figures 49 through 82 and the discussion in Section 3.2.5 "Summary of Field Corrected Blow Counts and Selected Laboratory Tests for Each Reach", we have plotted additional information (first for fine grained soils and then separately for coarse grained soils) are summarized below:

- Fine grained soils (exclusively), elevation vs. dry unit weight, moisture content and unconfined compressive strength.
  - Reach 4, Figure 84
  - Reach 5, Figure 85
  - Reach 6, Figure 86
  - Reach 7A, Figure 87
  - Reach 7B, Figure 88
  - Reach 8, Figure 89
  - Reach 14, Figure 90

Pocket penetrometer values were not included in these graphs.

- Fine grained soils (exclusively), overburden pressure vs. unconfined compression strength and dry unit weight
  - Reach 4, Figure 91
  - Reach 5, Figure 92
  - Reach 6, Figure 93
  - Reach 7A, Figure 94
  - Reach 7B, Figure 95
  - Reach 8, Figure 96
  - Reach 14, Figure 97
- Coarse grained soils (exclusively), effective overburden pressure vs. corrected blow count per foot, including plots of relative density based on Gibbs and Holtz (1957). Also includes effective overburden pressure vs. dry unit weight.

- Reach 4, Figure 98
- Reach 5, Figure 99
- Reach 6, Figure 100
- Reach 7A, Figure 101
- Reach 7B, Figure 102
- Reach 8, Figure 103
- Reach 14, Figure 104

- Coarse grained soils (exclusively), elevation vs. ϕ'
  - Reach 4, Figure 105
  - Reach 5, Figure 106
  - Reach 6, Figure 107
  - Reach 7A, Figure 108
  - Reach 7B, Figure 109
  - Reach 8, Figure 110
  - Reach 14, Figure 111

For Figures 105 through 111, there are 3 charts of elevation vs. ϕ'. The left hand chart is based on the Peck, Hanson and Thornburn (PHT, 1974) approach where the blow count is corrected for sampler size and overburden pressure; based on our engineering judgment we have selected a lower bound dotted line envelope for this method. The middle chart is based on the Schmertmann (1975) method where ϕ' is based on the relative density relationships shown in Figure 112; we have selected a lower bound dashed line envelope for this method. The right hand chart is a combination of the two methods; again the PHT dotted envelope and the Schmertmann dashed envelope are shown. Based on these two envelopes, we selected an intermediate "design curve" envelope shown on the right hand chart.

4.1.2.2 Granular Soils

Effective strength parameters were estimated by cross-referencing three methods. As an example, refer to Section Number 2 in Reach 4 near Boring SB 4-5.

In the first method, we estimated ϕ' values at different elevations from the right hand chart "design curve" envelope from Figure 105 for all of Reach 4.

In the second method (Schmertmann), we estimated relative density at specific depths based on corresponding N values and overburden pressure of granular layers in Boring SB 4-5. For example, between depths of 3 and 5 feet, the Schmertmann method relative density was determined from Figure 98 to be approximately 90% for this upper sand layer. From Figure 112 for this relative density, for fine sand $\phi'=39°$ and for coarse well graded sand $\phi'=45°$. From a practical viewpoint, the maximum ϕ' value is limited to 40°, regardless of the gradation and relative density.

In the third method, ϕ' was estimated based on the correlation of corrected blow count versus ϕ' (EM 1110-2-2502, 1989, Retaining and Flood Walls) for the PHT (1974) curve. For the silty gravel layer between depths of 3 to 5 feet, the second method yields $\phi'=33°$.

Based on the three methods, we selected $\phi'=33°$ for this layer for analysis.

4.1.2.3 Native Cohesive Soils

Undrained Strength

For fine grained soils exclusively for each reach, two sets of charts were developed for unconfined compression strength. For example for Reach 8, unconfined compression strength (Q_u) vs. elevation is shown in Figure 89 and Q_u vs. overburden pressure is shown in Figure 96. For a specific section number, we compared Q_u for the clay layers from the closest boring to the Q_u from Figures 89 and 96 for all of Reach 8. We then selected a design Q_u based on the range of plotted Q_u values. In this manner, the best estimate of Q_u was achieved. S_u was then estimated with $S_u=0.5Q_u$.

Drained Strength

A total of 47 triaxial circles resulted from CU tests performed (with pore pressure measurements) on relatively undisturbed cohesive soil samples in all six reaches. These test results are presented in Section 6 of Appendix D of Volume III and are plotted on Figure 83. Based on the data plotted on Figure 83, we estimate the average effective strength parameters for clay soils are about $\phi'=33.6°$ and $c'=191$ psf in all six reaches. Based on further evaluation of the CU test results in terms of unit weight and percent strain at failure, we then estimated ϕ' and c' on a reach by reach basis, as summarized in the following table.

Reach	Estimated Effective Shear Strength Parameters for Cohesive Soils from CU Tests	
	ϕ' (degrees)	c' (Cohesion, psf)
4	40	0
5	33	190
6	33	190
7	28	270
8	38	80
14	28	250

These specific parameters were then assigned to CL and CH layers in each of the reaches. For example, for Reach 7, we used $\phi'=28°$ and $c'=270$ psf.

Shear Strength for Rapid Drawdown

In accordance with Corps Engineer Manual, EM 1110-2-1902, Slope Stability, we have estimated shear strength parameters for the fine grained soils based on the improved method (Method G-3) developed by Lowe and Karafiath (1960), and modified by Wright and Duncan (1987), and Duncan, Wright and Wong (1990). The principal shear strength parameters include: 1) effective stress envelope (estimated from CU triaxial test results) and 2) undrained shear strength on failure surface during consolidation (also estimated from CU triaxial test results). Utilizing the 3-stage analysis capability of UTEXAS slope stability computer program, the appropriate shear strength parameters are estimated by interpolating between the strength parameter sets 1 and 2 for the effective normal stress at the potential slip surface. This method provides a rational approach to estimate undrained strength when the channel pool is drawn down faster than the pore water can escape; excess pore water pressures and reduced stability will result. In addition, this method accounts for more realistic shear strength in zones where drained strength is lower than the undrained strength. A summary of the two sets of shear strength parameters are presented in the table below.

Reach	Effective Stress Envelope		Undrained Shear Strength on Failure Surface during Consolidation (τ_{ff} versus σ'_{fc})	
	ϕ' (°)	c' (psf)	$\psi_{Kc=1}$ (°)	$d_{Kc=1}$ (psf)
4	40	0	24	450
5 and 6	33	190	31	90
7A and 7B	28	270	24	300
8	38	80	29	160
14	28	250	17	325

4.1.2.4 Compacted Cohesive Soils

Background

Laboratory tests were performed on a potential borrow material source from Boring SB 6-16 (Reach 6) between depths of 2 to 6 feet bgs. At this location, a cut is planned. The intent of the laboratory test program was to estimate shear strength properties of a fine-grained bulk sample that could possibly be used for fill where new levees are required in other parts of the project. The laboratory test results are summarized in Appendix E of Volume III. This sample was initially classified in the field as a sandy silt. However, based on subsequent laboratory test results, this sample was reclassified as a clayey sand with gravel. The laboratory test results are summarized as follows:

- Liquid limit is 26
- Plasticity index is 12
- Minus Number 200 sieve is 16 percent
- Plus Number 4 sieve is 35 percent
- Maximum dry density of 138 pcf
- Optimum moisture content of 7.5 percent
- Q_u of 580 psf at axial strain of 2.1 percent

- ϕ' of 44°
- c' of 0 psf

Although the laboratory tests on this material result in a classification as a granular soil (SC), we found that the material does possess some cohesion as indicated by the Q_u and plasticity index tests. Therefore we expect this material could be reused as levee fill; it may be necessary to blend in some lean clay to further increase the cohesion and decrease the permeability.

Undrained Strength

At the Morgan Hill Fire Station (see Appendix A of Volume II), clayey sand fill was encountered to a depth of about 3 feet bgs. The dry unit weight of 2 samples was about 116 pcf and Q_u ranged from 2,710 to 4,930 psf. Based on our experience at this location and with similar type projects, we estimate a S_u (one-half of Q_u) of about 1,500 psf and ϕ'=0° for design at ULFPP.

Drained Strength

We reviewed drained shear strength design parameters from several sources.

First, the Bureau of Reclamation (1987) has published average engineering properties of compacted soils on projects in the Western United States. For CL soils, they show a ϕ' of about 25° and a cohesion c' of about 1,480 psf (10.3 psi) for a dry unit weight of about 107 pcf. For ML soils, they show a ϕ' of about 34° and a c' of about 520 psf (3.6 psi) for a dry unit weight of about 99 pcf. For GC soils, they show a ϕ' of about 27½° and a cohesion of 1,470 psf (10.2 psi) for a dry unit weight of about 111 pcf.

Second, as discussed previously from Figure 83 (for overconsolidated undisturbed clays only), ϕ' is 33.6° and c' is 191 psf. Implicitly, onsite clays recompacted in the levee could have strength parameters equal to or greater than those in Figure 83. However, this may not be the case since they are overconsolidated, rather than normally consolidated.

Third, for the subsurface investigation for the Lower Guadalupe River Flood Control Project performed by URS for the District in 2002, the undrained shear strength parameters for lean clay fill (CL) were recommended to be ϕ'=24° and c'=450 psf (for dry unit weight less than 110 pcf) and ϕ'=33° and c'=0 psf (for dry unit weight greater than 110 pcf). For CL fill with a dry unit weight more than 110 pcf, the recommended values were ϕ'=33° and c'=0 psf.

Fourth, the United States Navy (NAVFAC DM-7.2, 1982) has published typical engineering properties of compacted materials. For CL soils, they show a ϕ' of about 28° and c' of about 270 psf (saturated sample).

In summary, we recommend that shear strength parameters for compacted CL or ML or GC soils be ϕ'=28° and c'=270 psf. These parameters were used in our analysis where compacted fill is required.

4.2 METHOD OF ANALYSIS

Four design conditions were analyzed for the slope stability evaluation in accordance with the Corps Manual for the Design and Construction of Levees (EM 1110-2-1913) dated April 30, 2000. The conditions, shear strength guidelines, and minimum factors of safety required for design are listed below. Left and right slopes looking upstream, were analyzed for each design condition.

Case	Design Condition	Required Minimum Factor of Safety
I	End of Construction	1.3
II	Sudden Drawdown	1.0
IIIa	Steady Seepage from Full Flood Stage with Design Flood Level Based on Corps Hydraulic Design Report	1.4
IIIb	Steady Seepage at "Zero Freeboard" Level at Crest of Levee or Bank (i.e., generally 3 feet above Design Flood Level)	Assumed 1.4
IV	Earthquake	Not applicable

Case IIIb is not included in the Corps Manual. However, District and Corps requested that Case IIIb be analyzed and the results are reported herein.

As outlined in the Corps guidelines, earthquake loadings are not considered in analyzing the stability of levees because of the low probability of an earthquake coinciding with a period of high water. Nevertheless, we evaluated the seismic stability of the levee because of the close proximity to major fault systems such as the San Andreas and Hayward.

- Initially performed pseudo-static slope stability analysis using a seismic coefficient of 0.15g for Case IV. If the factor of safety is greater than 1.0, no further analysis was required for seismic loading.
- If the factor of safety is equal to or less than 1.0 (for 0.15g for Case IV), URS then performed pseudo-static slope stability analysis to estimate the yield acceleration by first using a seismic coefficient approximately one-half of the median peak ground acceleration (PGA=0.3 to 0.33); this is consistent with the California Department of Conservation, Division of Mines and Geology, Special Publication 117, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," (1997).
- We then planned to perform simplified deformation analysis for the critical slip surface to estimate the magnitude of potential permanent deformation (Makdisi and Seed, 1977) based on the yield acceleration, estimated in the second step, and the estimated average maximum acceleration (a_{max}) along the slip surface.

The stability of the levees under Cases I, IIIa, IIIb and IV (pseudo-static) was evaluated using a limit equilibrium method based on Spencer's procedure of slices as coded in the program SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied. For Case II, we used UTEXAS3, which is also a limit equilibrium computer program based on Spencer's procedure, to evaluate the factor of safety against sudden drawdown loading. UTEXAS3 is coded with the 3-stage computation that is appropriate for the sudden drawdown loading condition.

4.2.1 Pore Pressures

The SEEP/W program was used to estimate the phreatic surface in the levee for the critical flood stage under the steady seepage condition based on the design water levels provided in the plans prepared by the Corps. The SEEP/W program and the seepage analyses are described in further detail in Section 6 "Seepage Evaluation" of this report. For a given cross section and design condition, the wetted face (phreatic surface) was estimated. The phreatic surface generated by SEEP/W was used to calculate the pore pressures within the levee for Cases II, IIIa and IIIb.

The phreatic surface at the steady seepage case is considered the worst-case as flood water in the channel establishes an equilibrium seepage pattern with the adjacent groundwater condition. However, it is our experience that transient seepage analysis based on a design hydrograph generally shows that the wetted face into the levee/channel bank never reached the height of the steady seepage condition; therefore the assumption of the piezometric surface is judged to be conservative.

4.2.2 Static Loading Conditions

The static loading conditions for slope stability analysis prescribed by the Corps are listed below.

- End of Construction
- Sudden Drawdown
- Steady Seepage from Full Flood Stage

A discussion of the conditions for analysis and appropriate strength parameters is found in the following sections.

End of Construction

The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for the pressures to dissipate. For this reason, undrained conditions exist for impervious embankment and foundation soils. Strength results from laboratory unconsolidated-undrained and unconfined compression tests were used for the fine grained soils and the effective friction angle was used for the granular soils, as they are assumed to be pervious and free draining.

Sudden Drawdown

The sudden drawdown case represents the condition whereby a prolonged flood stage saturates at least the major part of the waterside embankment and then drops faster than the soil can drain. This causes the development of excess pore pressures that may result in the slope becoming unstable.

A three-stage computation was used to estimate the strength of the cohesive soils during undrained loading. The first set of stability computations was performed to calculate stresses along the shear surface, which corresponds to the stresses prior to undrained loading. The second set of computations was performed to compute the factor of safety for undrained loading due to sudden drawdown. A third set of computations was performed only if the undrained shear strength employed in the second stage computations was greater than the shear strength that would exist if the soil were drained.

For this analysis, the effective stress envelope for cohesive soils is based on consolidated undrained test results discussed in Section 4.1.2.3 "Native Cohesive Soils". Effective stress parameters as discussed in Section 4.1.2.2 "Granular Soils", were used for granular soils throughout the analysis since these soils are assumed to be free draining. These strengths are in accordance with the Corps Manual.

The high water levels used at each section are the design high levels as specified on the plans prepared by the Corps. The long-term water levels reached during rapid drawdown were based on water levels measured in nearby borings and observation wells.

Steady Seepage from Full Flood Stage

The steady seepage case represents the condition where the water remains at or near full flood stage so that the embankment becomes fully saturated and a condition of steady seepage occurs. The design shear strength should be based on the effective stresses for both cohesive and granular soils; this should correspond to the residual strengths where previous shear deformation or sliding has occurred.

The water levels used are at the design high levels as specified on the plans prepared by Corps.

4.2.3 Earthquake Loading Conditions

The earthquake case represents the condition when the levee is subjected to earthquake motion. As discussed above, we performed a 3-step evaluation for the earthquake loading. In the first steps, a pseudo-static slope stability analysis was performed using a seismic coefficient of 0.15g for Case IV. If the factor of safety is greater than 1.0, no further analysis was required for seismic loading. If the computed factor of safety for a seismic coefficient of 0.15g was less than 1.0, then we performed a subsequent pseudo-static analysis to determine the failure surface for a seismic coefficient equal to one-half the peak horizontal ground acceleration as outlined in the 1997 Guidelines for Evaluating and Mitigation Seismic Hazards in California and as discussed in Section 2.4 of this report. This failure surface was then used in a third step to estimate permanent deformation of the slope under earthquake loading. The strength parameters used for the earthquake condition were the same as for the end of construction. The pseudo-static analysis assumes no strength gain associated with consolidation over time. Therefore, the short-term (undrained) strength determined during our field investigation is also used for the earthquake condition.

4.2.4 Tension Cracks

In our slope stability analyses, we assumed tensions cracks occurred only in cohesive soils to the bottom of the cohesive layer but to a depth no greater than the level of the design channel bottom. Furthermore, we assumed the following regarding tension cracks filled with water:

- For end of construction, tension cracks filled with water;
- For long term, no tension cracks; and
- For earthquake loading, tension cracks but not water.

No tensile strength is assumed for the upper layers of the soil stratum for short-term loading conditions. Therefore, appropriate tension cracks are assumed in the upper soil strata, particularly in the clayey soils. For long-term loading, the upper layers of the soil strata also assume no tensile strength.

4.3 STABILITY ANALYSIS RESULTS

The minimum (critical) factors of safety (FOS) computed using the various methods of analysis are summarized in the table below for the slope that are currently proposed.

	EOC (left)	EOC (right)	Sudden Draw-down (left)	Sudden Draw-down (right)	Steady Seepage IIIa (left)	Steady Seepage IIIa (right)	Steady Seepage IIIb (left)	Steady Seepage IIIb (right)	Pseudo-Static (left)	Pseudo-Static (right)
Cross Section No.	**Min F.S. = 1.3 Required**	**Min F.S. = 1.3 Required**	**Min F.S. = 1.0 Required**	**Min F.S. = 1.0 Required**	**Min F.S. = 1.4 Required**	**Min F.S. = 1.4 Required**	**Min F.S. = 1.4 Required**	**Min F.S. = 1.4 Required**	**Min F.S. = 1.0 Required**	**Min F.S. = 1.0 Required**
1	2.7	2.5	2.7	2.5	3.2	2.4	3.2	2.6	1.6	1.6
2	1.5	2.1	1.3	2.0	1.5	2.3	1.5	2.3	1.1	1.4
	-	-	2.6	2.6	-	-	-	-	-	-
3 Lower Slope	2.2	2.4	1.5	2.7	-	-	-	2.7	-	1.6
3 Overall	2.0	3.5	1.9	2.1	2.2	2.6	2.4	-	1.4	-
4	-	5.1	-	2.5	1.0**	4.6	-	4.6	-	2.4
5 Upper Slope	2.8	2.6	2.5	2.2	3.1	2.9	-	-	1.7	1.7
5 Lower Slope	2.9	2.8	3.0	2.2	4.4	2.7	4.4	2.8	1.8	1.8
6	2.1	0.9**	2.1	-	2.2	0.9**	2.5	-	1.4	0.7**
7*	2.5	1.9	2.5	-	2.7	2.7	2.7	2.5	1.6	2.0
8*	8.4	16	4.0	-	4.9	5.2	7.5	5.1	4.4	6.4
9	5.1	5.1	3.5	3.5	5.1	5.1	5.0	5.1	3.0	3.0
10	1.6	1.6	1.6	1.6	1.8	1.8	1.8	1.8	1.1	1.1
11	1.8	1.8	2.5	1.4	2.4	1.6	3.2	1.9	1.5	1.4
12	2.1	2.2	2.2	3.5	2.4	2.5	2.3	2.6	1.4	1.4
13 Overall	3.1	2.4	2.0	1.7	2.0	1.7	2.2	2.2	2.1	1.8
13 Shallow	0.6***	0.5***	-	-	-	-	-	-	-	-
14 Overall	1.7	1.8	1.2	1.2	1.8	1.4	1.8	1.8	1.5	1.4
14 Shallow	0.3***	0.2***	-	-	-	-	-	-	-	-
15	2.1	2.0	2.0	2.0	2.4	2.3	2.4	2.3	1.1	1.1
16	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	1.4	1.4
10A	1.6	1.8	1.6	1.8	1.6	1.7	1.7	1.9	1.2	1.3

EOC = End of Construction
Reported factor of safety rounded to nearest one-tenth.
*Right slope corresponds to outboard side; left slope corresponds to inboard side
**For FOS less than required minimum for proposed 1:1 slope; no additional cases analyzed
***Shallow slip surface through gabion slope face

The analyses are based on the slope geometry shown on Figures 113, 114 and 115; both the existing ground surface and design cross section are shown on these figures.

It should be noted that in cuts, the factor of safety was determined for the steady seepage condition, although in some cases (for example Reach 6, Section 4, Case III a, left) there is no outboard slope. Therefore, there is no emerging seepage on the outboard slope. Nevertheless, we included the slope stability results for completeness, recognizing these boundary/geometry conditions.

At Sections 3, 5 and 14, factors of safety were computed for upper and lower portions of the proposed embankment slopes.

The effect of existing adjacent lightly loaded structures on the left bank was included in the slope stability calculations for Sections 12 and 14. At each section, we assumed an exterior wall load of about 1,000 pounds per linear foot. In addition we assumed at Section 14 that the exterior wall is positioned 15 feet horizontally from the toe of the left wall.

The approximate locations of the critical slip surfaces are shown on figures in Appendix F of Volume IV.

As illustrated in the preceding table, the majority of the cross-sections that we analyzed have computed factors of safety that meet or exceed the minimums required by the Corps. Exceptions include Section 4 (left bank) and Section 6 (right bank) where the design slope of 1:1 was found to be unstable for the steady state seepage and end of construction cases, respectively. For these two cross-sections, no additional analyses were performed for the other design loading conditions. Based on the geometry and computed factors of safety for the opposite embankment at both sections, it appears that laying these relatively steep slopes back at an inclination of 3:1 (horizontal to vertical) should yield suitable factors of safety.

At Cross Sections 13 and 14 in Reach 8, the cross-sections provided to us indicate vertical to near vertical side slopes that we assumed would be supported with gravity retaining walls. For our analysis, we modeled the cross-sections using gabion walls and analyzed the global stability of the slope, looking at critical failure surfaces that were located around and beneath the wall. These results are shown in the first row of results for Sections 13 and 14. The second row for both of these sections shows the computed factor of safety for the end of construction case for a critical slip surface passing through the gabion wall. In each case, the factor of safety for failure surface through the gabion wall is below 1.0. Based on these results, we recommend that retaining structures at these locations be designed to satisfy conditions of internal and global stability. Please refer to Section 12.7 "Geotechnical Recommendations for Final Design" for further discussion and recommendations for Cross Sections 13 and 14. Case II, sudden drawdown, was not analyzed for these sections. This case should be analyzed once slope geometry, including a possible retaining wall, is determined in final design.

For Sections 1, 6, 7, 16, and 10A, the factor of safety shown for the sudden drawdown condition is reported equal to the end of construction case. At each of these sections, the assumed soil profile did not consist of any clay fills or native clay soils; therefore, a three-stage analysis was not required.

For Case IV seismic loading, all of the cases analyzed, with the exception of Sections 4 (left bank), 6 (right bank), 13, and 14, as described above, had computed factors of safety greater than 1.0. Therefore, no deformation analysis was performed

4.4 EROSION POTENTIAL CONSIDERATIONS

Based on Section 1.2 "Project Description and Understanding " the average channel velocity for each Reach is reported as follows.

Reach Number	Average Channel Velocity (feet per second)	Comments
4	4.8	
5	4.6	
6	4.5	
7A	4.0	
7B	3.0-4.0	
8	5.4	Between Station 147 + 35 and Station 178 + 70
8	2.8	Between Station 199 + 25 and Station 207 + 50
14	3.3	Between Station 0 + 00 and Station 10 + 00
14	5.0	Between Station 10 + 00 and Station 124 + 00
14	4.0	Between Station 124 + 00 and Station 180 + 00

Average channel velocities range from 2.8 to 5.4 feet per second (fps).

Anticipated soil conditions along the new channel alignment vary considerably, for example from fat clays, lean clays, clayey gravels to well graded gravels. From an erosion resistance viewpoint, fat clays and lean clays would be considered to be highly erodible whereas well graded gravels with little or no fines would be considered to have a low potential for erosion.

Although erosion control and slope protection measures are beyond the scope of our services, it can be inferred from the available data that (1) the soils along the channel are variable and (2) the soils are subject to water velocities on the order of 2.8 to 5.4 fps. Therefore, there is a potential for erosion of some soils along the alignment. This was confirmed particularly along portions of the north end of Reach 14 (near Corps Hydraulic Stations 114+00 to 122+00), where considerable erosion has been observed.

In our slope stability study, URS assumed there was no erosion of the bottom and sides of channel, including levee/berms. We recommend that the Corps/ District consider the effect of erosion during final design.

SECTION 5

A new levee is planned at six of the sections analyzed for slope stability in "Section 4 Levee Slope Stability Evaluation." Therefore, we estimated short-term elastic and long-term consolidation settlement at Sections 7 through 12 where height of new levee ranges from 1 to 5 feet. A copy of the settlement calculations are presented in Appendix G of Volume IV. We estimated elastic compression using elastic theory based on modulus values for clay soils (from unconfined compression strength) and granular soils (from SPT blow counts). To estimate consolidation settlement, we used the results of laboratory consolidation tests: the maximum past pressure was compared to the present insitu effective stress. Over consolidation ratios (OCR) ranged from 3 to more than 10. The following table summarizes our settlement estimates.

			Estimated Settlement			
Reach	**Section**	**Levee Height (feet)**	**Elastic Comp. (inch)**	**Elastic Comp. (mm)**	**Long Term Consolidation (inch)**	**Long Term Consolidation (mm)**
6A	7	5	0.3	7	0.0	0
6B	8	4.5	0.3	6	0.1	2
7A	9	1	0.0	1	0.1	2
7A	10	1.7	0.1	1	0.2	6
7B	11	3	0.1	3	0.2	6
7B	12	2	0.1	1	0.3	8

We estimate that elastic compression will occur in a short period of time, i.e., within 1 month after the levees are built; the estimated elastic compression ranges from 1 to 7mm (0.1 inch to 0.3 inch).

As shown in the table above, we estimate long-term consolidation settlement to generally range from 2 to 8mm (0.1 to 0.3 inch); the levees could be overbuilt to compensate for this long-term settlement. We estimate consolidation should be essentially complete in a period of 1 to 3 years.

SECTION 6

6.1 METHOD OF ANALYSIS

The flood levels used in the analysis of each of the selected sections were provided by the District. We understand the flood levels correspond to the Corps Design Flood Levels that are based upon the 100-year flood in Reaches 7 and 8, the 10-year flood in Reaches 5, 6 and 14 and the 5-year flood in Reach 4.

Seepage through the levee or into cut slopes during maximum flood stage was analyzed using a steady state analysis. A finite element program, SEEP/W [developed by Geo-Slope International (1992)], was used to perform calculations. SEEP/W is a computer program of two-dimensional code with isoparametric and higher order finite elements capable of solving saturated and unsaturated flow conditions using steady state and transient analysis. The program can represent multiple soil types with anisotropic hydraulic conductivity; it can also represent prescribed constant and transient boundary conditions.

The two boundary conditions established for the steady state analysis are constant head and no flow. The nodes beneath the water on the inboard side of the levee use the boundary condition of head equal to the height of the design water level. A boundary condition of no flow was established across the bottom of the finite element mesh. The nodes on the upstream portion of the levee were assigned a flux-type review flow boundary condition. In this case, after the heads are computed for all nodes, the review nodes are modified if any one of them has a computed head greater than the y-coordinate of the node (that is, if the porewater pressure is greater than zero). Using the maximum pressure review option, the review node with the maximum positive pressure head is identified by SEEP/W. The head at this node is then set to the y-coordinate for the next iteration. At the outboard boundary of the mesh, a constant head boundary was established. The constant head value was inferred from water levels observed in historical borings in the vicinity.

6.2 CROSS SECTIONS AND RESULTS OF ANALYSIS

It is expected the levees will be constructed of well compacted lean clays or clayey gravels. Some of the native soils directly underlying the future levee fills are lean to fat clays. Since the clayey soils are relatively impervious, it is our engineering opinion that (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. However, there are also locations where levees will be placed on granular soils or cuts will be made primarily in granular soils. Considering the variability of the soil conditions, we performed a steady-state seepage analysis on the 17 critical cross sections. These cross sections correspond to those listed and described in Section 4.1 "Cross Sections Selected for Analysis."

Since laboratory permeability tests were not included in the scope of services, the coefficients of hydraulic conductivity (permeability) for different types of soil were estimated from other laboratory tests, published data and field tests. They included broad empirical relationships between soil types and permeability (Cedergren, 1967) as shown in Figure 116 and correlation relationships based on grain size distribution as described in the Corps' EM-1110-2-1913 (COE, 1978) and the NAVAC DM-7.1 (1982).

For lean clay and fat clays, we have estimated a coefficient of permability of 10^{-6} centimeters per second (cm/sec) based on our local experience and Figure 116.

Based on the gradation of granular soils along most of the alignment, we estimated k values using the following formula developed by Hazen (1911).

$$k\ (\text{cm/sec}) = CD_{10}^{\ 2}$$

Where D_{10} = effective size in centimeters

$C = 100$

This formula should be considered as an approximation.

Using the above formula, we estimate the following order of magnitude k values:

- For gravels
  - k is about 10^{-2} cm/sec for fines content less than 10 percent
  - k is about 10^{-3} cm/sec for fines content between 10 and 20 percent
- For sands
  - k is about 10^{-3} to 10^{-4} cm/sec for fines content between 10 and 20 percent
  - k is about 10^{-5} cm/sec for fines content of about 40 percent

Falling head permeability tests were also performed by the District (2005) in June 2004 at Borings SB 7A-10 and SB 7A-12 in clayey gravel at a depth of about 10 feet bgs. The estimated coefficient of permeability was about 10^{-3} cm/sec.

The k value of 10^{-3} cm/sec (12 percent fines) estimated by the District compares well with the k value of 10^{-3} cm/sec by URS for gravels with 10 to 20 percent fines. Based on this information, we conclude an average k value of 10^{-3} cm/sec in granular materials is reasonable.

The ratio of anisotropy of the horizontal to vertical conductivity was assumed to be 1 for all soil types.

The approximate steady-state phreatic surface line is presented on figures in Appendix H of Volume IV for Cross Sections 1 through 16 and 10A. The SEEP/W output for the steady state condition at the seventeen (17) cross sections is presented and includes the estimated flow quantities in cubic feet per hour per linear foot for a steady state seepage condition with river levels at 100-year, 10-year or 5-year elevations, as summarized below.

Reach Number	Cross Section Number	Approximate Corps Design Station	Estimated Seepage Quantity for Corps Design Flood Level: Cubic Feet per Hour per Linear Foot (Left Side)	(Right Side)	Total	Gallons per Minute per Linear Foot (Total)
4	1	378+00	2.86E-01	1.25E-01	4.12E-01	5.14E-02
4	2	472+00	2.20E+00	4.45E+00	6.65E+00	8.30E-01
5	3	518+00	1.85E-01	1.87E-01	3.72E-01	4.64E-02
6	4	573+00	7.63E-01	3.73E-01	1.14E+00	1.42E-01
6	5	619+00	9.33E-01	9.20E-01	1.85E+00	2.31E-01
6	6	673+00	1.73E+00	1.55E+00	3.28E+00	4.09E-01
6 (Levee 6A)	7	23+20 (Levee Station)	N/A	2.80E+00	2.80E+00	3.49E-01
6 (Levee 6B)	8	7+50 (Levee Station)	N/A	2.14E+00	2.60E+00	3.25E-01
7A (Berm)	9	62+00	8.46E-03	1.10E-02	1.94E-02	2.42E-03
7A (Berm)	10	14+00	1.05E-01	1.06E-01	2.11E-01	2.63E-02
7B (Berm)	11	90+00	1.52E-01	2.70E-01	4.22E-01	5.26E-02
7B (Berm)	12	112+00	1.24E-01	1.11E+00	1.24E+00	1.54E-01
8	13	151+00	1.80E-01	1.49E-01	3.29E-01	4.10E-02
8	14	165+00	6.08E-02	5.57E-02	1.16E-01	1.45E-02
14	15	9+00	9.17E-01	4.40E-01	1.36E+00	1.69E-01
14	16	170+00	1.36E+00	1.40E+00	2.76E+00	3.44E-01
7A	10A	34+00	4.69E-01	4.87E-01	9.56E-01	1.19E-01

We also estimated steady state seepage flow quantities with river levels with no freeboard (zero freeboard) in these same seventeen (17) cross sections. These figures are also presented in Appendix H of Volume IV and the results are summarized in the table below.

Reach Number	Cross Section Number	Approximate Corps Design Station	Estimated Seepage Quantity for Zero Freeboard: Cubic Feet per Hour per Linear Foot			Estimated Seepage Quantity for Zero Freeboard: Gallons per Minute per Linear Foot (Total)
			(Left Side)	(Right Side)	Total	
4	1	378+00	4.28E-01	2.05E-01	6.33E-01	7.89E-02
4	2	472+00	2.54E+00	4.54E+00	7.09E+00	8.84E-01
5	3	518+00	2.62E-01	2.65E-01	5.27E-01	6.57E-02
6	4	573+00	9.34E-01	4.55E-01	1.39E+00	1.73E-01
6	5	619+00	1.11E+00	1.09E+00	2.20E+00	2.75E-01
6	6	673+00	1.55E+00	1.90E+00	3.45E+00	4.31E-01
6 (Levee 6A)	7	23+20 (Levee Station)	N/A	3.52E+00	3.52E+00	4.39E-01
6 (Levee 6B)	8	7+50 (Levee Station)	N/A	2.60E+00	2.60E+00	3.25E-01
7A (Berm)	9	62+00	1.30E-02	1.79E-02	3.09E-02	3.85E-03
7A (Berm)	10	14+00	1.37E-01	1.38E-01	2.76E-01	3.44E-02
7B (Berm)	11	90+00	2.10E-01	3.59E-01	5.68E-01	7.08E-02
7B (Berm)	12	112+00	1.13E-01	1.18E+00	1.29E+00	1.61E-01
8	13	151+00	1.91E-01	2.76E-01	4.67E-01	5.83E-02
8	14	165+00	7.22E-02	6.61E-02	1.38E-01	1.72E-02
14	15	9+00	9.17E-01	4.40E-01	1.36E+00	1.69E-01
14	16	170+00	N/A	2.05E+00	2.05E+00	2.55E-01
7A	10A	34+00	7.01E-01	7.31E-01	1.43E+00	1.79E-01

For Corps Design Flood Level, the estimated seepage quantity ranges from $2.42x10^{-3}$ to $8.30x10^{-1}$ gallon per minute (gpm) per linear foot. For a 30.5-meter (100-foot) long section, the corresponding total flow quantity (for Corps Design flood level) through the levee or into cut slopes at the above 17 sections is expected to range from about ¼ to 80 gallons per minute (gpm). In our opinion these seepage quantities range from small to large. However, it should be emphasized that the quantity of estimated seepage refers to the underground water flow through a theoretical cross-section at the levee toe or channel bank.

The above discussion is for a steady state condition such that seepage through the levee or into cut slopes achieves equilibrium during the 100-year, 10-year or 5-year flood elevation.

SECTION 7

7.1 FLOOD WALLS

7.1.1 Background

In Reach 8 in downtown Morgan Hill, between Station 147+35 (West Dunne Avenue) and Station 178+70 (Main Avenue), the proposed design (see Figure 117) consists of approximately 3,000 feet of trapezoidal vegetated gabion channel consisting of a 20-foot bottom width, 1V:0.5H side slopes, depth of 7.5 to 10 feet, and a maximum top width of 30 feet. The average design channel velocity in this section is 5.4 feet per second.

Since Reach 8 does not have the ability to convey the channel capacity plus freeboard in several locations, flood walls will be necessary at these locations. Between Station 151+00 and Station 175+50 approximately 1,700 feet of flood walls 1.5 to 2.5 feet high will be built on both sides of the creek. No other details are available for the flood walls.

7.1.2 Subsurface Conditions

Based on the above project description, we have reviewed the subsurface information between Station 151+00 to Station 175+50 (exclusive of borings at the intermediate box culverts), including the following borings (listed going from south to north ends): SB 8A-1, SB 8A-2, SB 8A-2D, SB 8A-3, SB 8A-3C, SB 8A-4, SB 8A-4D, SB 8A-4F, SB 8A-5 and SB 8A-5B. A review of these boring logs reveals two different soil profiles: (1) the South Segment generally includes a shallow layer of lean clay overlying predominately granular deposits and (2) the North Segment consists primarily of lean clays and fat clays. Each segment will now be discussed.

7.1.2.1 South Segment

The South Segment extends from about Station 151+00 to Warren Avenue (about Station 170+00) and includes Borings SB 8A-1,SB 8A-2, SB 8A-2D, SB A-3, SB 8A-3C, SB 8A-4, SB 8A-4D and SB 8A-4F. Fill, consisting of medium dense to dense clayey sand to hard fat clay, to depths of 2½ to 4 feet bgs in Borings SB 8A-1, SB 8A-2D, SB 8A-3C and SB 8A-4F was encountered; a pavement section was also noted in Borings SB 8A-2, SB 8A-2D and SB 8A-4D. Within the eight borings of the South Segment native granular deposits generally occurred including medium dense to very dense clayey gravel, poorly graded gravel, clayey sand and poorly graded sand to depths ranging from 23½ to 40 feet bgs; relatively impermeable greenstone was encountered at depths of 17½ to 32½ feet bgs in Borings SB 8A-3, SB 8A-3C, SB 8A-4D and SB 8A-4F. In general, some stiff to hard lean clay interbeds were also encountered.

Based on the gradations of the granular samples in eight borings in the South Segment, URS estimated k values using the following formula developed by Hazen (1911).

$$k\ (\text{cm/sec}) = CD_{10}^{\ 2}$$

$$\text{Where } D_{10} = \text{effective size in centimeters}$$

$$C = 100$$

This formula should be considered as an approximation.

Using the above formula, we estimate the following order of magnitude k values:

- For gravels
  - k is about 10^{-2} centimeters per second (cm/sec) for fines content less than 10 percent
  - k is about 10^{-3} cm/sec for fines content between 10 and 20 percent
- For sands
  - k is about 10^{-3} to 10^{-4} cm/sec for fines content between 10 and 20 percent
  - k is about 10^{-5} cm/sec for fines content of about 40 percent

Falling head permeability tests were also performed by the District (2005) in June 2004 at Borings SB 7A-10 and SB 7A-12 in clayey gravel at a depth of about 10 feet bgs. The estimated coefficient of permeability was about 10^{-3} cm/sec.

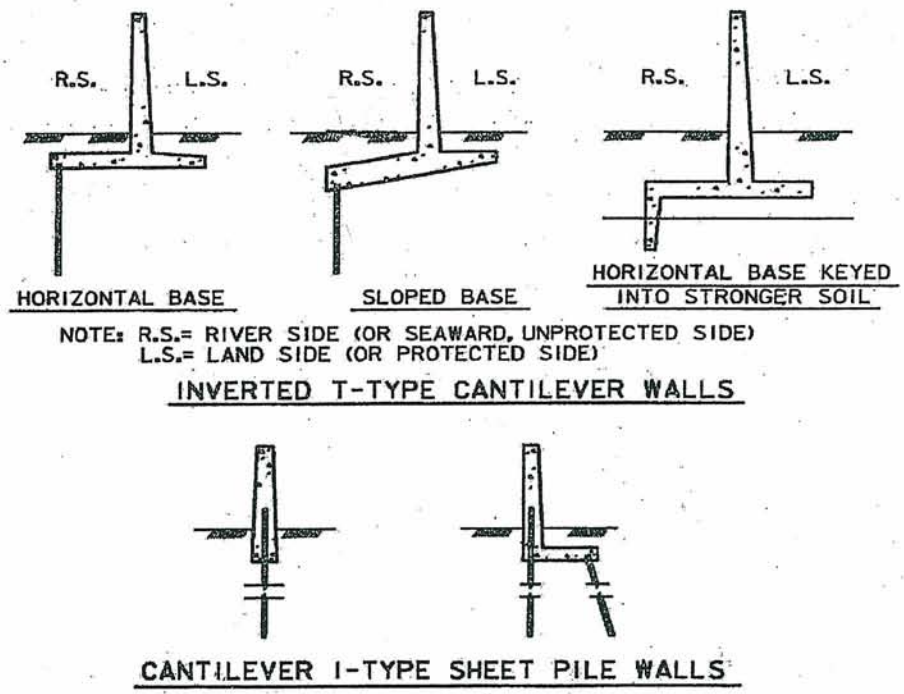
The k value of 10^{-3} cm/sec (12 percent fines) estimated by the District compares well with the k value of 10^{-3} cm/sec by URS for gravels with 10 to 20 percent fines. Based on this information, we conclude an average k value of 10^{-3} cm/sec in granular materials is reasonable for design in Reach 8.

7.1.2.2 North Segment

The North Segment extends from Warren Avenue (about Station 170+00) to about Station 175+50 and includes Borings SB 8A-5 and SB 8A-5B. Lean clay with an interbed of elastic silt, was observed in Boring SB 8A-5 to a depth of about 26½ feet bgs; a 5½-foot thick deposit of poorly graded sand occurred directly below with a deeper lean clay layer which extended to terminal depth of 40 feet bgs. Primarily lean clay deposits were observed in Boring SB 8A-5B to a depth of 17 feet bgs; these clay deposits were underlain by relatively impermeable greenstone to terminal depth of 29 feet bgs. Due to the numerous clay layers in these two borings, the subsurface soils in the North Segment are considered to be relatively impervious.

7.1.3 Corps Requirements

The recommendations presented in this section are consistent with the Corps Manual No. EM 1110-2-1913 (Design and Construction of Levees, April 30, 2000) and Corps Manual No. EM 1110-2-2502 (Engineering and Design, Retaining and Flood Walls, September 29, 1989). According to EM 1110-2-2502, the most common types of flood walls are cantilever T-type or cantilever I-type walls. Examples of these two wall types are shown below (from EM 1110-2-2502).



7.1.3.1 Cantilever T-Type Wall

Most flood walls are of the inverted T-type. The cross bar of the T serves as a base and the stem serves as the water barrier. When founded on soil, a vertical base key is sometimes used to increase resistance to horizontal movement. If the wall is founded on rock, a key is usually not provided. Where required, the wall can be supported on piles. A sheet pile cutoff can be included to control underseepage or provide scour protection for the foundation. T-type walls may be provided with a horizontal or sloped base.

7.1.3.2 Cantilever I-Type Wall

I-type flood walls consist of driven sheet piles capped by a concrete wall. I-walls are most often used in connection with levee and T-wall junctions or for protection in narrow restricted areas where the wall height is not over 8 to 10 feet, depending on soil properties and geometry.

7.1.3.3 Reach 8 Flood Walls

From a geotechnical viewpoint, the inverted T-type is recommended for the North Segment since foundation soils are primarily lean clays that are relatively impervious and do not require a cutoff wall.

At the South Segment, the subgrade soils are primarily granular with an estimated k value of order 10^{-3} to 10^{-4}cm/sec. According to the Corps EM 1110-2-1902 (Slope Stability) dated October 31, 2003, "As a rough guideline, materials with values of permeability greater than 10^{-4} cm/sec usually will be fully drained throughout construction." Since k is estimated to be about 10^{-3} to 10^{-4} cm/sec in the South Segment subgrade soils, consideration was given to installing a line of sheet piles underneath a cantilever I-type sheet pile wall. However, most of the granular soils encountered are dense to very dense and contain gravels. It would therefore be impractical (unfeasible) to drive sheet piles in these materials. One alternative is to install a line of overlapping auger cast piles, Deep Soil Mixing (soil-cement piles) or drilled piers to serve as a cutoff; however, this may be very expensive. A second alternative is to place the South Segment on inverted T-type cantilever walls and excavate a partial concrete cutoff wall. Assuming no cutoff wall, we estimate the (steady state) seepage loss to be approximately 10^{-2} gallons / minute / lineal foot of wall. Considering the brevity of the design hydrograph, steady state seepage may never be achieved and the flow rate could be less than that estimated. A third alternative is to replace the flood wall system with a reinforced concrete U-shaped channel with walls that provide adequate (3-foot) freeboard. Selection of an appropriate wall type should be determined by others before or during final design.

7.1.4 Wall Settlement

Within the limits of the proposed flood walls, there are 15 borings (SB 8A-1B through SB 8A-5B). At the south end and middle between Borings SB 8A-1B and SB 8A-4F, generally there is a surficial thin layer (2½ to 6½ feet thick) of stiff to hard lean clay or fat clay; to a depth of at least 20 feet bgs, the underlying soils are mostly very dense clayey sand and/or clayey gravel. At the north end at Borings SB 8A-5B and SB 8-5 the soils are primarily stiff to hard lean clay and fat clays to depths in excess of 20 feet bgs. We assume the existing ground surface will not be raised where flood walls are required. Therefore, long-term consolidation settlement of the high strength clays is expected to be nil. During passage of the design flood, the time duration is assumed to be relatively small and settlement of the flood wall is expected to be nil for two reasons: (1) the underlying sands/gravels are dense to very dense and (2) the clays are impermeable and will not consolidate in the short time duration.

7.1.5 Driving Side Design Earth Pressure

Consistent with the Corps requirements, a tension crack should be assumed to extend vertically down to the bottom of the shear key on the riverside. Therefore, the flood walls should be designed to resist an "at-rest" earth pressure equal to water with an equivalent fluid pressure of 1 Mg/m3 (62.4 pcf). This value is recommended, regardless of foundation type.

7.1.6 Resisting Side Design Earth Pressure

At Rest Earth Pressure

The estimated moist unit weight of the clay soil adjacent to the flood wall and the corresponding submerged unit weight of the same clay soil is 18.8 and 9.1kN per cubic meter (120 pcf and 57.6 pcf), respectively. Assuming that soil will not be excavated adjacent to the wall during its lifetime, at-rest conditions occur on the resisting side. For this at-rest condition, a coefficient of lateral earth pressure of 0.58 is recommended to estimate the pressure against the flood wall. Use the moist unit weight when groundwater is at the bottom of the footing at the landside edge. When groundwater rises to ground surface above the landside footing edge, use the submerged unit weight to estimate the equivalent earth pressures and then add the hydrostatic water pressure.

The wall resistance to sliding should be evaluated for both loading cases that represent different stages of seepage. The upper 300mm (1 foot) of embedment should be neglected for design.

We understand flood walls may be designed with a shear key. Where there is a shear key or the bottom of slab is inclined, the shear resistance through the clay subgrade should be estimated using a cohesion (undrained condition) of 191.5kPa (4,000 psf) or a friction angle (drained condition) of 25°, whichever is less.

In the event that a spread footing (horizontal base) with no shear key is selected, an ultimate coefficient of friction of 0.30 (tangent of 17 degrees for clay to concrete friction angle) should be used between the base of the flood wall and underlying soil, with the total resistance not to exceed 45kPa (950 psf). This assumes (1) that the flood walls are cast neat against undisturbed engineered fill, (2) the bottom of footing is horizontal and (3) there is no shear key.

Passive Pressure

In accordance with the Corps manual for certain loading conditions, resisting-side earth pressure may be increased, but not to exceed one-half the (ultimate) passive pressure using unfactored shear strengths. Again, assuming that soil will not be excavated to the wall during its lifetime, an ultimate passive resistance of the soil should be estimated using a coefficient of passive earth pressure of 2.5. With groundwater at the bottom of the footing at the landside edge, use the moist unit weight of soil. When groundwater rises to ground surface above the landside footing edge, use the submerged unit weight to estimate the passive earth pressure and then add the hydrostatic water pressure. The wall resistance to sliding should be evaluated for both loading cases that represent different stages of seepage. The upper 300mm (1 foot) of embedment should be neglected for design.

7.1.7 Spread Footing Foundations

We assume the bottom of the flood walls will be positioned about 3 feet below present ground surface in Reach 8. Soils encountered at these levels are dense poorly graded gravel and dense clayey gravel in the South Segment and stiff lean clay in the North Segment. For these subsurface conditions, spread footing foundations are recommended.

For spread footing foundations situated at levels described in the previous paragraph, on dense poorly graded gravel or dense clayey gravel in South Segment or lean clay in North Segment, the following design bearing pressures are recommended.

Flood Wall Location	Ultimate Bearing Capacity		Soil Type	Estimated Allowable Bearing Capacity		Recommended Allowable Bearing Capacity	
	psf	kPa		psf	kPa	psf	kPa
South Segment	7,500	359	Cohesionless and Cohesive	2,500	120	2,500	120
North Segment	7,500	359	Cohesive	2,500	120	2,500	120

The ultimate bearing capacity values in the above table are estimated using procedures outlined in EM 1110-1-1905 "Bearing Capacity of Soils" dated October 30, 1992. In addition, the estimated allowable bearing capacity was estimated using a factor of safety (FS) of 3 for cohesionless soils and cohesive soils; this is consistent with EM 1110-2-2502 "Retaining and Flood Walls" dated September 29, 1989 for retaining walls (Table 4-1) and inland flood walls (Table 4-2).

It is recommended that the bottom of footing be placed a minimum of 0.91 meter (3 feet) below lowest adjacent finished grade.

7.1.8 Foundation Uplift

A spread footing foundation will be temporarily subject to buoyant uplift during the 100-year flood event. We recommend the water pressure on the base be estimated by the line-of-creep method presented in the Corps Manual EM 1110-2-2502.

7.2 FISH BARRIER DESIGN

7.2.1 Background

Two fish barriers are planned, one in Reach 7A at Station 14+00 (or Corps Design Station 2+00) and the other in Reach 14 at Station 10+00. Each barrier consists of a vertical reinforced concrete wall that extends 2 to 5 feet above the design channel invert, as shown in Figure 118. No other design information is presently available.

Based on Corps Plan and Profile Drawings at approximate Station 2+00 in Reach 7A, we estimate the following approximate Elevations:

- Design channel invert about 304 feet
- Right and left levee crest at 319
- Ground surface at 314

Therefore, excavation depth is on the order of 10 feet.

According to the Corps Plan and Profile Drawings at Station 10+00 in Reach 14 and survey elevation of Borings SB 14-26 and SB 14-26B, we estimate the following approximate Elevation:

- Design channel invert about 236.7 feet at downstream side of fish barrier
- Design channel invert about 242.8 feet at upstream side of fish barrier
- Ground surface varies from 240 to 249

It appears excavation depth is about 7 feet at middle of channel.

7.2.2 Subsurface Conditions

Boring SB 7A-13 was drilled in Reach 7A near Station 14+00. In Reach 14, Boring SB 14-26B was drilled at the channel invert whereas Boring SB 14-26 was drilled near the hinge point near Station 10+00.

At Boring SB 7A-13 4 feet of loose to medium dense clayey sand was underlain by medium dense well graded sand to a depth of 8 feet bgs. Beginning at 8 feet bgs and extending to the terminal depth of 31½ feet is a layer of dense poorly graded gravel. Groundwater was encountered at a depth of about 15 feet bgs, or about 5½ feet below proposed channel invert.

In Reach 14 at SB 14-26, a layer of hard sandy lean clay was encountered from ground surface to a depth of 5 feet. The clay was generally underlain by granular layers ranging from medium dense to dense poorly graded sand, dense clayey gravel, medium dense clayey sand to the terminal boring depth of 39½ feet. Bottom of channel Boring SB 14-26B generally encountered mostly dense clayey gravel. Groundwater was measured about 18 feet below existing channel invert.

7.2.3 Spread Footing Foundations

We assume the bottom of the Fish Barrier will be positioned about 5 feet below design channel invert or about Elevation 299 and Elevation 232 in Reaches 7A and 14, respectively. Soils encountered at these levels are dense poorly graded gravel and dense clayey gravel in Reaches 7A and 14, respectively. For these subsurface conditions, spread footing foundations are advisable.

For spread footing foundations situated at Elevations described in the previous paragraph, on dense poorly graded gravel or dense clayey gravel, the following design bearing pressures are recommended.

Fish Barrier Location	Ultimate Bearing Capacity		Soil Type	Estimated Allowable Bearing Capacity		Recommended Allowable Bearing Capacity	
	psf	kPa		psf	kPa	psf	kPa
Reach 7A, Station 13+00	32,000	1,532	Cohesionless	10,000	479	5,000	239
Reach 14, Station 10+00	15,000	718	Cohesionless	5,000	239	5,000	239

The ultimate bearing capacity values in the above table are estimated using procedures outlined in EM 1110-1-1905 "Bearing Capacity of Soils" dated October 30, 1992. In addition, the estimated allowable bearing capacity was estimated using a factor of safety (FS) of 3 for cohesionless soils and cohesive soils; this is consistent with EM 1110-2-2502 "Retaining and Flood Walls" dated September 29, 1989 for retaining walls (Table 4-1) and inland flood walls (Table 4-2). In our engineering judgment, we have limited the maximum or "recommended allowable bearing capacity" to a value of 5,000 psf in the above table for cohesive and cohesionless soils.

7.2.4 Design Criteria

According to Corp guidelines (EM 1110-2-2502, September 29, 1989, Retaining and Flood Walls), the Fish Barrier walls should be evaluated for the following three loading conditions:

- Case R1, Usual Loading: the backfill is in place to the final elevation with surcharge load applied.

- Case R2, Unusual loading: same as Case R1, except other temporary loads are considered.
- Case R3, Earthquake Loading: same as Case R1 with the addition of earthquake loads.

The guidelines also outline the need for stability evaluation for three failure modes: (1) sliding at the base, (2) overturning of the wall and (3) bearing failure. These failure modes are shown graphically on Figure 4-1 of the Corps manual EM 1110-2-2502.

For the free draining foundation soils encountered at both fish barrier walls, the evaluation was based on effective strength parameters. We also assumed the footing subgrade will not be disturbed during construction.

7.2.5 Lateral Pressures

Soil lateral pressures for static loads (Cases R1 and R2) were estimated for the driving and resisting sides of the wall. For the driving side lateral pressure, an at-rest backfill condition (k_0) was assumed, as recommended by the guidelines (USACE, 1989). This at-rest lateral pressure coefficient was estimated using Jaky's equation (1944) and then compared with Coulomb active earth pressure theory modified with a Strength Mobilization Factor (SMF) of 2/3; the higher value is selected for design. For the resistant side lateral pressure, a passive soil condition was assumed without strength modification (SMF = 1.0). The following summarizes the soil parameters and lateral soil coefficients we recommend for the three loading conditions.

- Saturated unit weight = 20.4kN per cubic meter (130 pcf)
- Submerged unit weight = 10.6 kN per cubic meter (67.6 pcf)
- At-rest coefficient $k_0 = 0.44$
- Passive pressure coefficient $k_p = 3.5$
- Sliding friction factor f = 0.35

Soil lateral pressure for earthquake loads (Case R3) should be estimated using the general wedge earthquake method. This method is outlined in Paragraph 3-26, Sub-paragraph c of the Corp manual (page 3-65 of the EM 1110-2-2502). We assume that these retaining walls are to be evaluated for the 84^{th} Percentile Earthquake ground motion. The following presents the seismic coefficients that are the horizontal accelerations acting on the seismic wedge.

- At Reach 7A, $k_h = 0.50g$
- At Reach 14, $k_h = 0.56g$

7.3 BRIDGE DESIGN

7.3.1 Background

A new 3.5m (11.5 foot) wide driveway bridge is planned in Reach 7A at about Station 114+70 (or Corps Design Station 10+300). No other bridge design information is presently available such as geometry, abutment/bent layouts and structural loads. However, we assumed the bridge will be supported on two abutments and a center bent.

According to the Corps Plan and Profile Drawings at Corps Design Station 10+300, we estimate the following approximate elevations.

- Design channel invert 321 feet
- West levee crest 332 feet
- East levee crest 331 feet
- Ground surface varies from 328 to 330 feet

Therefore, channel excavation depth is on the order of 7 to 9 feet bgs.

7.3.2 Subsurface Conditions

Two Borings SB 7B-4E (east abutment) and SB 7B-4F (west abutment) were drilled at locations on opposite sides of the creek centerline (see Figure 18).

At Boring SB 7B-4E, mostly very stiff to hard lean clay was noted from ground surface to 16 feet bgs; however between depths of 4 to 8½ feet, hard fat clay was encountered. Very dense clayey gravel was encountered between depths of 16 and 29 feet bgs. A thin (1 foot) interbed of hard sandy clay occurred between depths of 29 and 30 feet. Below this interbed is a very dense clayey sand which extended to the terminal depth of 40 feet.

At Boring SB 7B-4F, medium to hard sandy lean clay fill was encountered to a depth of 5 feet, underlain by native dense clayey sand to 8½ feet bgs. Occurring directly below the clayey sand is stiff lean clay to a depth of 13½ feet bgs. Mostly clayey sand (with gravel) was encountered from 13½ feet to the terminal depth of 40 feet; it is generally very dense. A ½ foot thick stiff lean clay interbed was observed at 29 feet.

Boring SB 7B-4F was dry to the terminal depth of 40 feet, whereas groundwater was encountered at 22 feet bgs in Boring SB 7B-4E. The latter corresponds to a depth of about 15 feet below design channel invert.

7.3.3 Foundation Type

We assume the bottom of the abutment foundations and the bent foundation will be positioned about 5 feet below present ground surface (say Elevation 323) and 5 feet below design channel bottom (say Elevation 316), respectively. Soils encountered at these levels are hard fat clay and hard lean clay in SB 7B-4E and dense clayey sand to stiff lean clay in SB 7B-4F. Groundwater level was about 15 feet below channel invert at the time of drilling in Boring SB 7B-4E.

Considering these subsurface conditions, we expect either spread footing foundations or drilled piers would be feasible. Both foundation types will be discussed in the following section.

7.3.4 Spread Footing Foundations

For spread footing foundations situated at Elevations ranging between 316 (bent) and 323 (abutments), on stiff to hard clays or dense clayey sand, the following design bearing pressures are recommended.

Bridge Location in Reach 7A	Ultimate Bearing Capacity		Soil Type	Estimated Allowable Bearing Capacity		Recommended Allowable Bearing Capacity	
	psf	kPa		psf	kPa	psf	kPa
About Station 114+70	12,000	575	Cohesionless and Cohesive	4,000	191	4,000	191

The ultimate bearing capacity values in the above table are estimated using procedures outlined in EM 1110-1-1905 "Bearing Capacity of Soils" dated October 30, 1992. In addition, the estimated allowable bearing capacity was estimated using a factor of safety (FS) of 3 for cohesionless soils and cohesive soils; this is consistent with EM 1110-2-2502 "Retaining and Flood Walls" dated September 29, 1989 for retaining walls (Table 4-1) and inland flood walls (Table 4-2).

At the location of Boring SB 7B-4F sandy lean clay fill was encountered. Either footings should be positioned below existing fill on native soils, or existing fill should be removed and replaced with new engineered fill compacted to a minimum relative compaction of 95 percent.

Spread footings should extend to a minimum depth of 0.91m (3 feet) below the lowest adjacent finish grade, for the following reason. For the east abutment near Boring SB 7B-4E, a fat clay (CH) layer was encountered between depths of 4 to 8½ feet bgs. Although CH soils are potentially expansive clays, the footings could be supported directly on the stiff to hard native CH clays. In order to minimize the potential effects of shrinking and swelling of the expansive clays due to seasonal moisture change, it is recommended that the footings extend to a minimum depth of 0.91m (3 feet) below the lowest adjacent finished grade.

During final design when structural loads and the foundation layouts are available, URS should review the above recommendations. Furthermore, at that time the settlement can be estimated.

7.3.5 Drilled Piers (CIDH Piles)

As an alternative to spread footings, drilled cast-in-place concrete pier foundations (also referred to as cast-in drilled hole piles) can be used to support the abutments and/or bent; these piers can also resist lateral and uplift forces. Pier design capacities will be developed through skin friction. End bearing should be neglected. We recommend an ultimate skin friction value of 47.9kPa (1,000 psf) be used to compute the required embedment length of straight shaft drilled piers to support axial compression and tension (uplift) loads. As a guideline, we recommend pier diameter be at least 30 inches.

Design load (dead plus live) capacities for both downward and uplift loading should be determined using a factor of safety of 2.0. Selection of the factor of safety for seismic loading conditions is left to the Structural Engineer.

The drilled pier foundations recommended for support of the proposed structure will be capable of resisting lateral loads. The magnitude of the lateral load resistance is dependent on many factors, including the pier size and embedment length, condition of pier cap fixity, the physical properties of the surrounding materials, and the yield moment capacity of the pier. During final design, P-Y curves for the selected shaft diameter and stiffness can be developed when structural loads and foundation layouts are known.

Drilled piers/CIDH piles should be constructed in accordance with the requirements of Section 49-4 of the Caltrans Standard Specifications, latest edition. If piers extend below approximately Elevation 312, deposits of mostly very dense clayey sand and clayey gravel are anticipated. Moreover, groundwater has been found at about Elevation 306 in Boring SB 7B-4E; it could be higher depending when the piers are installed (such as late spring). Therefore, it should be anticipated that drilling would require the use of temporary casing or slurry to prevent caving during construction. If casing is used and the holes are dewatered, the casing should be withdrawn from the hole slowly as the concrete is being placed; a minimum head of concrete of 1.5m (5 feet) should be maintained above the bottom of casing at all times. Alternatively, excavating and depositing concrete under slurry could be considered. For this option, the Contractor should submit a detailed placing plan at least 72 hours in advance that outlines the materials and procedures to be employed. Concrete deposited under slurry should be carefully placed in a compact, monolithic mass and by a method that will prevent washing of the concrete. It should be a continuous operation lasting not more than the time required for each concrete placing operation at each pier, as submitted in the placing plan. All drilled piers should be installed under the direct observation of the Geotechnical Engineer.

7.3.6 Lateral Pressures on Abutment Walls

During final design when layouts of abutment walls are available, the design lateral pressures can be calculated using the data from this report.

SECTION 8

8.1 CULVERT DESCRIPTION

Eight (8) new culverts are planned in Reach 7 and Reach 8, as summarized in the tables below.

Reach 7 Location	Boring Numbers	Proposed Design							Existing	
		Roadway Width (ft)	Culvert Size #-w (ft) x h (ft)	From Station (ft)	To Station (ft)	Length (ft)	Type of Crossing	Approximate Invert Elevation (ft)	Roadway Width (ft)	Culvert Size w (ft) x h (ft)
Spring Avenue	SB 7B-1 Exist SB 7B-2 Exist	60	3-10x9	133+30	133+90	60	RCB*	330	60	
West Dunne to Ciolino Avenue (Morgan Hill Plaza)	SB 7B-3 Exist Through SB 7B-7 Exist	-	1-8x8	147+70	141+00	674	RCB	331	-	

*RCB-Reinforced Concrete Box

Reach 8 Location	Boring Numbers	Proposed Design							Existing		
		Roadway Width (ft)	Culvert Size #-w (ft) x h (ft)	From Station (ft)	To Station (ft)	Length (ft)	Type of Crossing	Approximate Invert Elevation (ft)	Roadway Width (ft)	Culvert Size w (ft) x h (ft)	Type of Crossing
5th Street	SB 8A-1B	60	2 - 10x9	152+60	152+00	60	RCB	332	60	5x5	RCB
4th Street / Monterey Hwy	SB 8A-2B SB 8A-2C SB 8A-2D	270	2 - 10x9	157+00	154+30	270	RCB	332 to 333	270	9x6	RCB
3rd Street	SB 8A-3B	14	2 - 10x9	160+70	161+30	60	RCB	334	14	14x7	RCB
2nd Street / Del Monte Avenue	SB 8A-4B SB 8A-4C SB 8A-4D	250	2 - 10x9	165+00	167+50	250	RCB	335 to 336	250	10x5	RCB
Warren Avenue	SB 8A-4F	40	2 - 10x9	170+00	170+50	50	RCB	337	40	10x5	RCB
Main to Wright along Hale Ave (Future Santa Teresa Expwy)	SB 8A-6 SB 8A-6B SB 8A-7 SB 8A-7B SB 8A-8 SB 8A-8B SB 8A-9	40	2 - 10x8	178+70	199+25	2,200	RCB	339 to 343	N/a	-	-
Main Street	-	N/a	-	-	-	-	-	-	70	9x5	RCB
Wright / Hale Avenue	-	N/a	-	-	-	-	-	-	110	60"	RCP**

**Reinforced Concrete Pipe

8.2 SUBSURFACE CONDITIONS

Based on our review of the logs of borings for each box culvert listed in the above tables, we have summarized the expected soil and groundwater conditions to bottom of culvert slab. Details regarding the structural pavement sections at the boring locations are noted on the logs. We assume the culvert floor slab is about 1 foot thick.

8.2.1 Spring Avenue

Sides of excavation to a depth of about 9½ feet bgs are expected to encounter stiff to hard lean clay. Subgrade soils (at 9½ feet) are generally medium dense to dense clayey sand and silty sand. Depth to groundwater ranges from 6½ to 7 feet bgs in nearby borings, or about 3 feet above subgrade. Shallowest groundwater at nearby piezometer OW 7B-2 was measured at 5.6 feet bgs (on March 9, 2005), or about 4 feet above subgrade.

8.2.2 Morgan Hill Plaza

Sides of excavation to depths of about 9 to 11 feet bgs are expected to encounter highly variable soil conditions ranging from stiff to hard lean clay to medium dense to dense poorly graded sand, well graded sand and clayey gravel. A wide range of subgrade soils (at 9 to 11 feet) is also anticipated from stiff lean clay to medium dense to dense silty sand, well graded sand and clayey gravel. Depth to groundwater ranges from 8 to 20 feet bgs in nearby borings; at Borings SB 7B-3, SB 7B-4 and SB 7B-5, groundwater is about 1½, 1 and 3 feet, respectively above subgrade. Shallowest groundwater at nearby piezometer OW 7B-3, OW 7B-5 and OW 7B-7 was measured at depths of 4.6 feet bgs, 5.9 feet bgs and 5.5 feet bgs, all on March 9, 2005, or about 4½ to 5 feet above subgrade.

8.2.3 5th Street

Sides of excavation to a depth of about 8 feet bgs are expected to be loose to dense clayey sand fill. Subgrade soils (at 8 feet) are dense poorly graded sand overlying dense poorly graded gravel. Depth to groundwater is about 24 feet in nearby boring, or 16 feet below subgrade. Shallowest groundwater at nearby piezometer OW 8A-1B was 6.8 feet on March 14, 2006, or about 1 foot above subgrade.

8.2.4 4th Street / Monterey Highway

Sides of excavation to a depth of about 10 feet bgs are expected to be variable soils; stiff lean clay fill or dense clayey sand fill to a depth of 3 feet underlain by stiff sandy lean clay, and dense clayey gravel or dense to very dense clayey sand. Subgrade soils (at 10 feet) are dense poorly graded sand or medium dense to dense clayey sand. Depth to groundwater in nearby borings ranges from 23½ to 34 feet bgs, or more than 10 feet below subgrade. Shallowest groundwater at nearby piezometer OW 8A-2C was 7.9 feet on January 20, 2006, or about 2 feet above subgrade.

8.2.5 3rd Street

Sides of excavation to a depth of about 9 feet bgs are expected to be dense to very dense clayey sand fill to 5 feet underlain by native very dense native clayey sand. Subgrade soils (at 9 feet) are expected to be native very dense clayey sand. Boring SB 8A-3B was dry to a depth of 32½ feet, or more than 10 feet below subgrade. Shallowest groundwater at nearby piezometer OW 8A-3B was 8.2 feet bgs on March 14, 2006, or about 1 foot above subgrade.

8.2.6 2nd Street / Del Monte Avenue

Sides of excavation to a depth of about 9 feet are anticipated to be medium sandy lean clay fill or native medium to stiff sandy lean clay underlain by mostly medium dense or dense clayey sand; a deeper very stiff lean clay layer was encountered in Boring SB 8A-4D. Subgrade soils are anticipated to be medium dense to dense clayey sand and very stiff lean clay. Depth to groundwater in nearby borings ranges from 20½ feet (Boring SB 8A-4B) to dry (depth of 24 to 33½ feet), or more than 10 feet below subgrade. Shallowest groundwater at nearby piezometer OW 8A-4 was measured at 6.6 feet bgs (on March 14, 2006), or about 1½ feet above subgrade.

8.2.7 Warren Avenue

Sides of excavation to a depth of about 7 feet bgs are expected to be hard fat clay fill to 3 feet underlain by hard elastic silt. Subgrade soils are anticipated to be medium dense clayey sand. Since Boring SB 8A-4F was dry to terminal depth of 36 feet, groundwater is more than 10 feet below subgrade. Shallowest groundwater at nearby piezometer OW 8A-5 was measured at 9.0 feet bgs (on March 14, 2006), or about 2 feet below subgrade.

8.2.8 Hale Avenue

Sides of excavation to a depth of about 7 to 9 feet bgs are expected to be primarily stiff to hard elastic silt and stiff to very stiff fat clay with a few interbeds of stiff to very stiff lean clay and medium dense silty sand. Subgrade soils are anticipated to be either medium dense clayey sand (Borings SB 8A-6 and SB 8A-6B) or stiff to hard lean clay. Depth to groundwater ranges from 17 to 39 feet bgs, or more than 9 feet below subgrade.

8.3 DESIGN APPROACHES

There are several different methods for determining vertical pressure and lateral pressure on reinforced box culverts. These methods include State of California Department of Transportation (Caltrans), American Association of State Highway and Transportation Officials, Inc. (AASHTO) and Corps. Design parameters for each of these methods will be discussed in the following paragraphs. Final selection of which criteria should be used will be based on a discussion with the District. The method selected may depend on the government agency that has jurisdiction. For example, we understand the box culvert at Hale Avenue will eventually be positioned below future Santa Teresa Expressway (under possible Santa Clara County jurisdiction) and the box culvert at Monterey Highway is under Caltrans jurisdiction. Both agencies may require using Caltrans design criteria.

8.4 CALTRANS CRITERIA

8.4.1 Background

There has been considerable research performed by different agencies on loading of various types of culverts, including reinforced concrete box culverts. The majority of this research has been performed by the California Department of Transportation, Caltrans, (Bacher et al, 1980, 1982, 1983, and 1990) with design criteria presented in Caltrans Bridge Design Specifications (February 1998). Based on long-term monitoring of instrumentation of box culverts, which included strain gauges and soil load cells, their research indicates soil loads increase over a period of 2 years or more. They found that the lateral pressures could be as much as the vertical pressures. Therefore, the traditional loading (Case 1) wherein the lateral pressure is taken as the active pressure (about ¼ to ⅓ of the vertical pressure) has been supplemented with a second loading (Case 2) wherein the lateral pressure is taken as about 70 percent of the vertical pressure. According to Caltrans (Bacher and Friedman, 1990), these two loadings are applied separately and the resulting maximum moments are utilized in design. The full lateral pressure 15.7 kN per cubic meter (100 pcf) condition also satisfies the saturated fill condition. Based on these considerations, we recommend the following geotechnical design parameters.

8.4.2 Static Loading Conditions

The reinforced concrete box culvert structures should be designed for two distinct static loading conditions: Case 1 (traditional loading) and Case 2 (supplemental loading), as discussed below.

For Case 1, the culvert should be designed for the following load combinations:

- Walls should be designed to resist an equivalent lateral fluid pressure of 5.5 kN per cubic meter (35 pcf)
- Roof should be designed to resist a column of soil with a moist unit weight of 22.0 kN per cubic meter (140 pcf)

For Case 2, the culvert should be designed for the following load combination:

- Walls should be designed to resist an equivalent lateral fluid pressure of 15.7 kN per cubic meter (100 pcf)
- Roof should be designed to resist a column of soil with a moist unit weight of 22.0 kN per cubic meter (140 pcf)

8.4.3 Traffic Loading Conditions

No additional surcharge pressure due to traffic loading should be applied to the walls.

All reinforced concrete box culvert roofs with less than 10 feet of cover should be designed for the following two conditions:

- Two feet of earth cover with HS 20-44 live load
- Ten feet of earth cover

It is recommended that impact loads be applied only to the roof slab of the box culvert, with the magnitude shown as follows:

- For earth cover of 1 foot or less, use 30 percent impact
- For earth cover of 1 to 2 feet, use 20 percent impact
- For earth cover of 2 to 3 feet, use 10 percent impact
- For earth cover greater than 3 feet, no impact

No impact load is recommended for the culvert invert. Consideration should also be given to structural design of the box culvert if loaded construction equipment is expected to pass over the top.

8.4.4 Earthquake Loading Conditions

According to Caltrans (1990), "observations of all types of underground structures in the 1971 San Fernando earthquake area and in the 1989 San Francisco (Loma Prieta) earthquake area, affirmed the cushioning effect the soil has on the performance of an underground structure during an earthquake. There were no failures due to an increase in soil pressures. Underground structures must move with the surrounding soil during earthquakes and usually will be supported by the interacting earth against crushing or collapse even if the structure joints are strained. If the earth does fault across a culvert, the tremendous forces will shear the submerged structure regardless of how the structure was designed. In special cases where underground structures are in soft ground (bay mud), consideration should be given to providing longitudinal structural continuity."

Considering the upper bound (100 pcf) of the two bands of lateral pressure recommended in Section 8.4.2 entitled "Static Loading Conditions", and the discussion in the previous paragraph, design of the box culvert walls and roof for increased loads during earthquake shaking is not considered necessary.

8.5 AASHTO CRITERIA

8.5.1 Static Loading Conditions

The box culvert structures should be designed for two distinct static loading conditions: Case 1 (traditional loading) and Case 2 (supplemental loading), as discussed below.

For Case 1, the culvert should be designed for the following load combinations:

- Walls should be designed to resist an equivalent lateral fluid pressure of 4.7 kN per cubic meter (30 pcf)
- Roof should be designed to resist a column of soil with a moist unit weight of 18.9 kN per cubic meter (120 pcf)

For Case 2, the culvert should be designed for the following load combination:

- Walls should be designed to resist an equivalent lateral fluid pressure of 9.4 kN per cubic meter (60 pcf)

- Roof should be designed to resist a column of soil with a moist unit weight of 18.9 kN per cubic meter (120 pcf)

8.5.2 Traffic Loading Conditions

Distribution of wheel loads through earth fills should be designed as follows:

- When depth of fill is 2 feet or more, concentrated loads should be considered as uniformly distributed over a square with sides equal to 1¾ times the depth of fill
- When such areas from several concentrations overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab.
- For single spans, the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between faces of end supports or abutments.
- When the depth of fill is less than 2 feet, the wheel load shall be distributed as in slabs with concentrated loads.

8.6 CORPS CRITERIA

The Corps does not recommend specific design criteria for lateral earth pressure on the box structures, except that horizontal pressures are controlled by the backfill requirements (Conduits, Culverts and Pipes, EM 1110-2-2902, October 31, 1997). Therefore, we estimated the lateral pressure induced by the dead (earth) load using project backfill soil information based on the principles of soil mechanics. The proposed box culverts located in Reaches 7B and 8 are positioned in variable soil types, ranging from clayey sand (SC) fill at Boring SB 8A-1B to fat clay (CH) in Boring SB 8A-6B. For that reason, we estimated the lateral soil pressure for both soils types and selected the maximum soil pressure (based on CH soils). The lateral soil pressures were developed for a long-term condition associated with an at-rest backfill stress condition using the consolidated soil properties. With regard to groundwater, the shallowest (long-term) water depths measured in Reaches 7B and 8 are 3.7 and 4.6 feet bgs in piezometers OW 7B-4B (March 14, 2006) and OW 7B-3 exist (March 9, 2005), respectively. To estimate the exterior water pressure on the box culverts we assumed the groundwater rose to the ground surface. Based on these considerations, we recommend the following geotechnical design parameters.

8.6.1 Static Loading Conditions

The reinforced concrete box culvert structures should be designed for the following load combinations:

- Walls should be designed to resist an equivalent lateral fluid pressure of 16.1 kN per cubic meter (103 pcf)
- Roof should be designed to resist a column of soil with a moist unit weight of 20.4 kN per cubic meter (130 pcf)

8.6.2 Traffic Loading Conditions

- Roof should be designed to resist a uniform surface live load of 300 psf
- Walls should be designed to resist an equivalent uniform pressure of 180 psf

No impact load is recommended for the culvert invert. Consideration should also be given to structural design of the box culvert if loaded construction equipment passes over the top.

8.6.3 Earthquake Loading Conditions

- Earthquake loads were not included.

8.7 DESIGN BEARING PRESSURES

As discussed in Section 8.2 "Subsurface Conditions" the bottom of most of the proposed box culverts (subgrade) will be situated in variable soils ranging from clayey sand (SC), silty sand (SM), poorly graded sand (SP), well graded sand (SW), clayey gravel (GC) and poorly graded gravel (GP). Along the Hale Avenue box culvert subgrade soils are expected to be either clayey sand or lean clay.

Based on our geotechnical analysis of specific subsurface conditions, the following design bearing pressures are recommended.

Box Culvert Location	Ultimate Bearing Capacity		Soil Type	Estimated Allowable Bearing Capacity		Recommended Allowable Bearing Capacity	
	psf	kPa		psf	kPa	psf	kPa
Spring Avenue	50,000	2,394	Cohesionless	16,600	795	5,000	239
Morgan Hill Plaza	11,000	527	Cohesive	3,600	176	3,600	176
5th Street	50,000	2,394	Cohesionless	16,600	795	5,000	239
4th Street / Monterey Highway	11,000	527	Cohesive	3,600	176	3,600	176
3rd Street	50,000	2,394	Cohesionless	16,600	795	5,000	239
2nd Street / Del Monte Avenue	11,000	527	Cohesive	3,600	176	3,600	176
Warren Avenue	50,000	2,394	Cohesionless	16,600	795	5,000	239
Hale Avenue	17,400	833	Cohesive	5,800	278	5,000	239

The ultimate bearing capacity values in the above table are estimated using procedures outlined in EM 1110-1-1905 "Bearing Capacity of Soils" dated October 30, 1992. In addition, the estimated allowable bearing capacity was estimated using a factor of safety (FS) of 3 for cohesionless soils and cohesive soils; this is consistent with EM 1110-2-2502 "Retaining and Flood Walls" dated September 29, 1989 for retaining walls (Table 4-1) and inland flood walls (Table 4-2). In our engineering judgment, we have limited the maximum or "recommended allowable bearing capacity" to a value of 239kPa (5,000 psf) in the above table for cohesive and cohesionless soils. It should be noted that the allowable bearing pressures assume the culverts will bear directly on undisturbed soils.

8.8 CONSTRUCTION CONSIDERATIONS

8.8.1 General

The proposed construction is expected to require excavation and some dewatering. The following sections address some of the construction considerations related to the box culverts.

8.8.2 Temporary Construction Excavations

Excavations for the proposed box culverts are anticipated to be about 7 to 11 feet deep. The excavations could be completed with sloping sides without shoring. Safety standards set by OSHA limit the height of unshored vertical excavations to 5 feet if construction personnel will be working in the excavations. The latest (1989) set of guidelines published by OSHA classify soils in detail as either Type A, B, or C. In general, Type A soils are stronger, Type B soils are intermediate, and Type C soils are weaker. Based on the soil type, depth, duration the excavation is open, and the sequence of soils exposed in the excavation, OSHA recommends maximum allowable slopes. For example, for excavations in homogeneous soils 20 feet or less in depth, they state that maximum allowable slopes (horizontal to vertical) should be ¾ to 1, 1 to 1, and 1½ to 1 for Type A, B, and C soils, respectively. The soils encountered at the site are variable and correspond to OSHA Types A, B, and C. On this basis we recommend the following temporary side slopes.

Box Culvert Location	OSHA Soil Type	Recommended Temporary Side Slopes (H:V)
Spring Avenue	A	¾ : 1
Morgan Hill Plaza	C	1½ : 1
5th Street	C	1½ : 1
4th Street / Monterey Highway	B	1 : 1
3rd Street	B	1 : 1
2nd Street / Del Monte Avenue	B	1 : 1
Warren Avenue	A	¾ : 1
Hale Avenue	B	1 : 1

We recommend that the District determine the effect of excavation widths (based on the above table) on streets that need to be maintained operational during construction.

8.8.3 Dewatering

As discussed in Section 8.2 "Subsurface Conditions", the subgrade (bottom of excavation) is expected to be below groundwater by 0.3m to 1.5m (1 to 5 feet) at all box culvert locations except at Hale Avenue; this assumes shallowest groundwater measured in nearby piezometer reoccurs. Therefore, we anticipate some dewatering will be required to maintain a dry excavation at most box culvert locations. Dewatering probably can be accomplished with trenches and sump pumps. In any case, the Contractor should install a dewatering system that results in a dry excavation. In the event of rainfall at any of these locations (including Hale Avenue), seepage into the excavation probably can be controlled with trenches and sump pumps.

SECTION 9

9.1 BACKGROUND

In order to estimate structural pavement thicknesses for roads at a minimum of three new box culvert locations, URS collected six (6) bulk (disturbed) samples of soil at subgrade level from cuttings from a hollow stem drill rig and then submitted the samples to Signet Labs for R-Value testing. Three (3) R-Values also were performed by the District in Reach 7B. The locations where the nine (9) soil samples were collected are shown on Figure 119. The URS laboratory test results are presented in Appendix D of Volume III (Section 8) and both URS and District results are summarized in the following table.

Reach Number	Boring Number	Approximate Station (feet)	Depth (feet)	R-Value	Soil Type
5	SB 5-1	525+40	0-2	14	Sandy lean clay (CL)
6	SB 6-7	675+80	4-7	11	Poorly graded sand (SP-SM) fill with silt and gravel
7A	SB 7A-2	92+40	0-5	11	Sandy lean clay (CL) fill
7B	SB 7B-1 exist	133+50	1-5	10	Lean clay (CL)
7B	SB 7B-3 exist	141+20	1-5	61	Poorly graded sand (SP)
7B	SB 7B-6 exist	146+00	1-5	8	Lean clay (CL)
8A	SB 8A-2C	156+00	2-5	19	Sandy lean clay (CL) fill
8A	SB 8A-8B	194+40	2-5	5	Fat clay (CH)
14	SB 14-21	42+00	0-2	8	Sandy lean clay (CL)

The R-Values range from 5 to 19, except for an R-Value of 61 for SP soil at Boring SB 7B-3. Since SP soils do not represent the likely subgrade conditions, we have neglected the latter value in our analysis.

9.2 RECOMMENDED PAVEMENT SECTIONS

Our analysis was performed in accordance with Chapter 600 of California Department of Transportation, "Highway Design Manual", Fifth Edition, 1995 (Caltrans). Based upon the subgrade soil conditions described in the above table and our analyses, the recommended pavement structural sections are shown in Table 17 for R-Values of 5, 8, 10, 11, 14, 15 and 19 and Traffic Indices (TI) values of 4, 4.5, 5, 5.5 and 6, as requested. Furthermore, the pavement design sections presented later in this section are based on the R-Values tests (except 61), plus an R-Value of 15. The reason for including an R-Value of 15 is as follows. For imported soil on Caltrans projects, an R-Value of 15 typically is specified. Therefore, where imported soil is required on this project for roadway subgrade, it would be appropriate to specify a minimum R-Value of 15. In determining the structural pavement sections, we used an R-Value of 78 and 50 for Class 2 Aggregate Base and Class 4 Aggregate Subbase, respectively; these values are consistent with Caltrans standards. In Table 17, the design alternatives are provided with and without aggregate subbase each R-Value.

New pavements will be required at the following eight (8) new box culvert locations; the recommended design R-Values are also shown.

Reach Number	Box Culvert Location	Boring Number	Proposed Design R-Value					
			From Station (ft)	To Station (ft)	Length (ft)	Actual R-Value	Recommended Design R-Value	Soil Type
7	Spring Avenue	SB 7B-1 Exist, SB 7B-2 Exist	133+30	133+90	60	10	10	CL
7	West Dunne to Ciolino Avenue (Morgan Hill Plaza)	SB 7B-3 Exist Through SB 7B-7 Exist	147+70	141+00	674	8	8	CL
8	5th Street	SB 8A-1B	152+60	152+00	60	-	19	SC
8	4th Street / Monterey Hwy	SB 8A-2B, SB 8A-2C, SB 8A-2D	157+00	154+30	270	19	19	SC, CL
8	3rd Street	SB 8A-3B	160+70	161+30	60	-	19	SC
8	2nd Street / Del Monte Avenue	SB 8A-4B, SB 8A-4C, SB 8A-4D	165+00	167+50	250	-	19	CL
8	Warren Avenue	SB 8A-4F	170+00	170+50	50	-	5	CH
8	Main to Wright along Hale Ave (Future Santa Teresa Expwy)	SB 8A-6, SB 8A-6B, SB 8A-7, SB 8A-7B, SB 8A-8, SB 8A-8B, SB 8A-9	178+70	199+25	2,200	5	5	MH, CH

As shown in the above table, pavements at new box culverts along Warren Avenue and Hale Avenue can be designed using an R-Value of 5; however, this could be increased if imported fill is used with a design R-Value of 15. At these two locations the native soils are primarily fat clay (CH), which is susceptible to shrinkage and expansion with moisture changes; by hauling the CH offsite and replacing it with non-expansive material, the pavement surface will be less susceptible to seasonal movement. Similarly, a higher design R-Value of 15 for imported fill could be used at Spring Avenue and Morgan Hill Plaza in Reach 7.

The recommended structural pavement section can be selected from Table 17 using the recommended design R-Values from the above table.

SECTION 10

A general discussion of liquefaction potential is presented in Section 2 "Site Geology and Seismicity". The potential for liquefaction along the project alignment was evaluated for the actual subsurface conditions encountered using semi-empirical methods to compare shear stresses induced by earthquakes with those required to cause liquefaction. For this analysis (refer to Section 2.4 "Ground Motions In Project Area"), we used a peak horizontal ground acceleration (PGA) based on the 84^{th} percentile listed in Tables 1 through 15 for Site Numbers 4-A through 14-O. The locations of the Site Numbers are shown on Figure 33. Consequently, our evaluation was based on a moment magnitude M_w of 7.1 on the Calaveras fault resulting in PGAs ranging from 0.62g to 0.77g. A design high groundwater was selected along the alignment using the highest groundwater level (1) measured during field explorations in 2003, 2004 and 2005 and (2) measured in piezometers in 2003 through 2006. Prior to our detailed analyses, we reviewed all boring logs; we then screened any borings where (1) the soil was granular, i.e. sand and gravel, (2) $(N_1)_{60}$ was 40 blows per foot or less and (3) the layers were below design high groundwater. Based on these criteria, a total of 44 sections were selected for analyses; the spread sheets for these 44 sections are presented in Appendix I of Volume IV.

The clay layers along the alignment are not susceptible to liquefaction. Therefore, the potential for liquefaction of various sand and gravel layers at the site was evaluated. On the basis of our analysis, it appears that there is some potential for liquefaction to occur at twenty (20) boring locations presented in the following table.

Boring	PGA (g)	Depth of Potentially Liquefiable Layer (feet)	Estimated Post-Liquefaction Ground Surface Settlement (inches)	Comments
SB 4-4	0.72	37 to 40	0.4	Lateral spreading unlikely
SB 4-14	0.77	5 to 8	0.3	
SB 4-15	0.77	5½ to 8½ 35 to 37	0.9	For deeper layer, lateral spreading unlikely
SB 4-16	0.77	4½ to 10½	1.5	
SB 5-4	0.72	23½ to 27	0.9	Lateral spreading unlikely
SB 6-1B	0.66	6 to 8	0.3	
SB 6-9	0.69	7½ to 13½	1.0	
SB 6-11	0.69	8½ to 13½	0.9	
SB 6-21B	0.69	17 to 22 37 to 40	1.2	For deeper layer, lateral spreading unlikely
SB 7A-11B	0.62	36 to 40	1.0	Lateral spreading unlikely
SB 7B-4C	0.63	8 to 12	0.5	
SB 7B-6B	0.62	37 to 40	0.5	Lateral spreading unlikely
SB 8A-2D	0.62	8 to 13½	0.5	
SB 8A-6B	0.62	17½ to 22	0.4	
SB 14-2	0.70	25½ to 32	1.1	Lateral spreading unlikely
SB 14-8	0.72	12 to 18	1.1	
SB 14-14	0.74	27½ to 32	0.6	Lateral spreading unlikely
SB 14-17	0.74	6½ to 10	0.7	
SB 14-19	0.74	12 to 17	0.8	
SB 14-26	0.74	21 to 27	0.8	Lateral spreading unlikely

These layers range in thickness from 0.6 to 2.0 m (2 to 6½ feet). Should liquefaction occur, settlement could result at these locations. Settlement of the ground surface is estimated to be on the order of 8 mm to 38 mm (0.3 inch to 1.5 inches) at these locations. Estimates of these potential settlements were made using the procedures outlined by Tokimatsu and Seed, (1984).

Based on explorations conducted, some layers of potentially liquefiable soil were encountered below depths of at least 7.6m (25 feet) below ground surface including Borings SB 4-4, SB 5-4, SB 7A-11B, SB 7B-6B, SB 14-2, SB 14-14 and SB 14-26 listed in the above table. Since these potentially liquefiable layers are relatively deep, it is our engineering opinion that lateral spreading is unlikely at these locations. Therefore, consideration was given to the possibility of lateral spreading in the shallower granular layers at Borings SB 4-14, SB 4-15, SB 4-16, SB 6-1B, SB 6-9, SB 6-11, SB 6-21B, SB 7B-4C, SB 8A-2D, SB 8A-6B, SB 14-8, SB 14-17 and SB 14-19 (also listed in the above table). Consequently, we performed post-liquefaction slope stability analysis at two boring locations (Borings SB 6-9 and SB 7B-4C) using residual strengths estimated for the potentially liquefiable sand layers. Both circular and wedge shaped slip surfaces were included in the analyses. The results of the above analysis are presented below.

Boring Number	Approximate Station	Factor of Safety End of Construction			Residual Undrained Shear Strength (psf)
		Minimum Required	Left	Right	
SB 6-9	662+00	1.3	3.1	3.4	300
SB 7B-4C	120+00	1.3	1.6	1.6	300

The factors of safety against undrained loading simulating conditions immediately after liquefaction are all above 1.3. Therefore, we conclude that lateral spreading is unlikely at the two liquefaction prone locations evaluated. During final design, we recommend that this same analysis be performed to further evaluate the potential for lateral spreading at Borings SB 4-14, SB 4-15, SB 4-16, SB 6-1B, SB 6-11, SB 6-21B, SB 8A-2D, SB 8A-6B, SB 14-8, SB 14-17 and SB 14-19.

SECTION 11

11.1 GENERAL

All site preparation and earthwork should be done under the observation of a representative of the Geotechnical Engineer and in accordance with the recommendations presented herein. In general, grading/earthwork will include widening the channel, raising the crown and side slopes of existing levees, constructing new levees, excavating and backfilling for new flood walls, excavating for new tieback walls and culverts, constructing depressed maintenance roads and ramps, and removal of sediment in the channel.

11.2 CLEARING AND DEMOLITION

All concrete flatwork, existing pavements, aggregate base, armoring aggregate, loose stone, drainage lines, underground utilities, trees, brush, vegetation, fencing and abandoned structures designated for removal should be demolished and removed from the site. Abandoned utilities encountered during grading should be removed in their entirety.

11.3 GRUBBING

Grubbing consists of the removal, within the levee foundations or beneath flood walls, maintenance road and ramp areas, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectionable matter. Roots or other intrusions over 38mm (1½ inches) in diameter within the levee foundations or beneath flood walls and maintenance road areas should be removed to a depth of 900mm (3 feet) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation areas. Materials resulting from grubbing operations should be removed from the site. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill consisting of material similar to adjoining, soils should be replaced in thin lifts up to the final foundation grade.

11.4 STRIPPING

After foundation clearing and grubbing operations are complete, stripping should be commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil, including but not limited to the side slopes and crown of existing levees, the landside and waterside toe of existing levees, and the footprint of new levees and maintenance roads and ramps. The depth of stripping should be determined by local conditions and normally is expected to vary from 150 to 300mm (6 to 12 inches). The final depth of stripping should be determined by a representative of the Geotechnical Engineer during site preparation. Materials resulting from stripping operations should be removed from the site. Organic topsoil that does not contain debris may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. The stripped materials should not be re-used as engineered fill or blended with other materials.

11.5 REMOVAL OF SEDIMENT IN CHANNEL

The channel sediment areas should be excavated as shown on the drawings to bring those areas to their finish channel elevations. All sediment materials should be removed from the site and should not be re-used for construction of levees, maintenance roads, ramps and backfill for flood walls. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect.

11.6 REMOVAL OF EXISTING FILL

Existing fill was encountered locally in borings in Reaches 6, 7, 8 and 14 and varied considerably in texture from granular to cohesive. None of the fill is believed to have been engineered. Within the footprint of any new levee, these materials should be completely removed. In our opinion, the majority of these soils can be reused as new engineered fill, as approved by the Geotechnical Engineer in the field.

11.7 EXCAVATION

After the areas to receive fill have been properly cleared, grubbed and stripped, it is recommended that existing loose fills or soft deposits be removed to the underlying native soils and replaced with engineered fill. Flood wall areas should be excavated as required to bring those areas to their finish subgrade elevations. All loose soil and existing fill should be removed from the exposed subgrade in the flood wall locations prior to wall construction.

It is expected that excavation can generally be accomplished with conventional earthmoving equipment. However, within Reach 8 at Borings SB 8A-3, SB 8A-3B, SB 8A-3C, SB 8A-4B, SB 8A-4C, SB 8A-4D, SB 8A-4F and SB 8A-5B greenstone was encountered at depths ranging from 5.2 to 11.6 m (17 to 38 feet) bgs. In general, the SPT sampler was advanced in this greenstone material by driving 50 blows in a few inches. This material appears to be below bottom of most channel excavations in Reach 8. However, it is conceivable that shallower greenstone could be encountered during excavation. In that event, heavy-duty excavation equipment such as a Caterpillar 245 is expected to be capable of ripping and excavating the greenstone; consequently chunks or blocks may be generated after excavation.

11.8 SUBGRADE PREPARATION

After demolition, stripping and subgrade excavations are complete, the exposed surface to be filled should further be prepared by scarifying the top 150mm (6 inches) of the surface, moisture conditioning the material to near the optimum content, and recompacting to a minimum relative compaction of 90 percent of its maximum dry density in accordance with ASTM Test Designation D 1557. This procedure is intended to provide a tight bond between the foundation and fill, and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to minimize the risk of saturation from rainfall.

At some locations, such as the toe of existing levees or depressed maintenance roads, the subgrade surfaces will be positioned near or below the free groundwater level or in areas which have been subjected to standing water. The exposed materials are expected to consist of low plasticity clays, silts and fine sands.

Control of the water in most shallow excavations in these materials probably can be accomplished by sloping them to drain and pumping from sumps within the excavated areas. After dewatering, some of the subgrade surfaces might be stable enough to support construction equipment; whereas, other areas might develop pumping or rutting. Please refer to Section 11.12 "Construction Dewatering", for further discussion regarding this subject.

In order to minimize the potential for pumping subgrade conditions, it is recommended that the first lift of the fill be "track-walked" in place to a thickness of about 300mm (1 foot) using a bulldozer or other track-mounted equipment. The initial lift should be dry of the laboratory optimum and compacted to a minimum relative compaction of 90 percent. Rubber tired equipment, such as scrapers or bottom dump trucks, should not be permitted to traverse the area. Pumping or rutting will be further exacerbated by repeated passes of rubber tired equipment. This could occur wherever a silt or clay subgrade is encountered in the existing fill or native soil.

In the event that deep seated pumping develops, it is recommended that the unstable subgrade areas be "bridged" using a combination of stabilization fabric (Mirafi 600X or equivalent) covered by a layer of granular, bridging material. The bridging material should consist of "bank run" or "crusher run" material conforming to the gradation discussed in Section 11.9.3 "Bridging Material." Since bridging material is highly permeable, seepage should be expected within this layer. Therefore, bridging material should only be used in areas where seepage is acceptable.

After the reinforcing fabric has been placed on the exposed subgrade surface, the bridging material should be track-walked into place over the fabric. It is estimated that a 450mm to 600mm (18 to 24 inches) thick layer of bridging material probably will be needed. Rubber tired equipment should not be permitted to traverse pumping areas until placement of the bridging material and stabilization fabric has been completed. Then additional fill can be placed and compacted to provide desired finished grades.

In areas where the silt or clay subgrade pumps or ruts (in fill or native soil), lime treatment is an alternative treatment to achieve stabilization. Since lime stabilization results in a relatively impervious soil, it can be used anywhere in the levee cross sections. Where subgrade areas are silty sand or clayey sand, they may require a combination of quicklime and flyash stabilization. The Contractor should retain his own testing laboratory to perform tests to determine the appropriate lime and/or flyash content; this should include unconfined compressive strength tests. It has been our experience on other local projects that 4 to 5 percent quicklime by weight will generally achieve the desired stabilization results. All lime/flyash stabilized subgrade should be prepared in accordance with Section 24 "Lime Stabilization" of the July 1999 Caltrans Standard Specifications. The Contractor should retain a Specialty Subcontractor experienced with lime stabilization. Furthermore, the Contractor should include this work in his bid price.

11.9 FILL MATERIALS

11.9.1 Approval

Soil materials, whether from sources on or off site, should be approved by a representative of the Geotechnical Engineer for the intended use and specifically for a required location or purpose. It is expected that most materials will be imported.

11.9.2 Onsite Materials

Channel Sediment

Where it is planned to remove portions of the channel (sediment), these materials should not be re-used as engineered fill. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. Otherwise, these materials should be removed from the site.

11.9.3 Excavation and Reuse of Materials

11.9.3.1 General

The existing fill and native soils are composed of highly variable soil types: lean clays, fat clays, elastic silts, clayey sands, clayey gravels, poorly graded sands and well graded sands. As described in the following paragraphs, the majority of these materials could be reused as new levee fill providing they are properly processed to achieve a moisture content near the optimum prior to placement. The drying of materials with excessive moisture content should be in accordance with the requirements in Section 11.10 "Fill Placement and Compaction". However, excavation for box culverts, flood walls and channel widening will require some selectivity to be exercised if these materials are to be considered for reuse as possible new levee fill. A more detailed discussion is presented in the following paragraphs.

11.9.3.2 Flood Walls

New flood walls are planned in Reach 8 in downtown Morgan Hill between approximately Station 151+00 and Station 175+50. As described previously in Section 7.1.2 "Subsurface Conditions", a review of the boring logs in this area reveals two different soil profiles: (1) the South Segment (about Station 151+00 to 170+00) generally includes a shallow layer (less than 4 feet bgs) of lean clay overlying predominately granular deposits and (2) the North Segment (about Station 170+00 to 175+50) consists primarily of lean clays and fat clays.

In the South Segment, if these excavated materials are to be considered for reuse as possible new levee fill, some selectively will be required. For example, the lean clays could be placed to meet the requirements of "General Levee Fill" described later. In addition, the majority of granular soils could be thoroughly mixed with the lean clays to be reused as "Select Levee Fill" described later.

In the North Segment, the soils to be excavated are primarily fat clays and elastic silts with some lean clays; this includes some fill. In particular if fat clays and elastic silts are reused in new levees, shallow surface sloughing or slaking can occur as a result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. For that reason, the fat clays or elastic silts should either be (1) hauled offsite and wasted or (2) thoroughly mixed with granular soil to meet the requirements of "Select Levee Fill" described later. However, the lean clays could be selected and reused for backfill along the flood walls.

11.9.3.3 Box Culverts

New box culverts are planned in portions of Reach 7B and Reach 8 between Spring Avenue and Wright Avenue in Morgan Hill.

Below the pavement section between approximately Warren Avenue (Boring SB 8A-4F) and Wright Avenue (Boring SB 8A-9) the soils to be excavated are primarily fat clays and elastic silts with some lean clays; this includes some fill. In particular if fat clays and elastic silts are reused in new levees, shallow surface sloughing or slaking can occur as a result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. For that reason, the fat clays or elastic silts should either be (1) hauled offsite and wasted or (2) thoroughly mixed with granular soil to meet the requirements of "Select Levee Fill" or "General Levee Fill" described later.

Between approximately Warren Avenue (Boring SB 8A-4D) and Spring Avenue (Boring SB 7B-1) below the pavement section, the soils to be excavated range considerably from lean clay to poorly to well graded sand, clayey sand and clayey gravel; this includes some fill. If these materials are to be considered for reuse as possible new levee fill, some selectively will be required. For example, the lean clays could be placed to meet the requirements of "General Levee Fill" described later. In addition, the majority of granular soils could be thoroughly mixed with the lean clays to be reused as new levee fill.

Furthermore, the above recommendations for the reuse of soils generated from excavation for box culverts also apply to excavations for open channel between approximately Spring Avenue and Wright Avenue.

11.9.3.4 Channel Widening

Outside of Reach 8 (discussed above), the channel will be widened and /or deepened including Reaches 4, 5, 6, 7A, 7B and 14. At present, excavation quantities are not available for the different reaches. In particular Reach 7A is a new diversion channel, resulting in presumably large quantities of excavated material.

As discussed in Section 3.2.4.4 "Reach 7A" along Reach 7A (Borings SB 7A-13 through SB 7A-1), some of the borings (5 total) encountered primarily granular layers beginning at ground surface to depths of 4½ to 30 feet; whereas the remainder of the borings (14 total) encountered lean clay and fat clay to depths of 3 to 17½ feet bgs. The latter were mostly underlain by granular layers. If these excavated materials are to be considered for reuse as possible new levee fill, some selectively will be required. For example, the lean clays could be placed to meet the requirements of "General Levee Fill" described later. In addition, the majority of granular soils could be thoroughly mixed with the lean clays to be reused as new "Select Levee Fill".

With the exception of Reach 7B, the soils in the upper 20 feet in Reaches 4, 5, 6 and 14 are primarily granular; in a few locations lean clay was encountered to variable depths. It would appear that there is an excess of granular soil in these areas that would not meet the requirements of "General Levee Fill". Where onsite lean clay is encountered (which will be relatively limited) the lean clays could be mixed with granular materials to meet the requirements of "General Levee Fill" or "Select Levee Fill". Alternatively, surplus excavated lean clay and/or fat clay from Reach 8 could be hauled to these reaches and mixed with onsite granular materials to be reused as new "Select Levee Fill".

In Reach 7B subsurface conditions are variable, and it is difficult to generalize. Nevertheless, most borings reveal a surficial layer of cohesive soil to depths of 2 to 8½ feet bgs underlain by alternating layers of granular soils and cohesive soils. Again, it would appear that there is an excess of granular soil in these areas that would not meet the requirements of "General Levee Fill". Where onsite lean clay is encountered (which again will be relatively limited), the lean clays could be mixed with granular materials to meet the requirements of "General Levee Fill" or "Select Levee Fill". Alternatively surplus excavated lean clay and/or fat clay from Reach 8 could be hauled to these reaches and mixed with onsite granular materials to be reused as new levee fill.

Existing Armoring Aggregate

Where there is an existing surficial layer of armoring aggregate (on the crown, i.e. crest), it should be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope. This material could be re-used for surfacing new maintenance roads, provided it is not contaminated or mixed with fine-grained clay or silt soils.

11.9.4 Import Materials

General Levee Fill

It is recommended import materials used for levee construction consist of a soil or soil-rock mixture free from organic matter or other deleterious substances. The material should contain at least 50 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity, as classified by the Unified Soil Classification System, having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 150mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 64mm (2½ inches). Sand and gravel size particles should comprise no more than 50 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite.

Select Levee Fill

Select levee fill for the outer levee shell should consist of material free from organic matter and substantially free of shale or other soft, poor durability particles. It should not contain slag aggregate or recycled materials, such as glass, shredded tires, portland cement concrete rubble, asphalt concrete rubble, or other objectionable material as determined by the Engineer. The material should contain no more than 30 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity, as classified by the Unified Soil Classification System, having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 150mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 60mm (2½ inches). Sand and gravel size particles should comprise a minimum of 70 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite.

Bridging Material

Bridging material, for bridging soft subgrades for maintenance road areas, should consist of a reasonably well graded mixture of coarse angular gravel and cobbles. The following gradation limits are recommended as a general design guideline.

Sieve Size		Percentage Passing
(inches)	(mm)	
6	150	100
2	50	0 – 50
¾	19	0 – 10
Durability Index: 25 minimum		

Armoring Aggregate

Armoring aggregate for placement on the crown of the levees, maintenance roads and ramps, should consist of clean, hard and durable gravel or crushed rock, conforming to the following gradation.

Sieve Size			Percentage Passing
(inches)	(mm)		
3	75		100
1½	37.5		75-55
¾	19		60-40
½	12.5		45-30
3/8	9.5		35-20
¼	6		25-15
No. 200	No. 200		20-0

The rock for armoring aggregate should have a minimum specific gravity of 2.40 in accordance with California Test Method No. 206 and a minimum durability index of 35 in accordance with California Test Method No. 229.

11.10 FILL PLACEMENT AND COMPACTION

All levee fills should be spread in uniform horizontal lifts not exceeding 200mm (8 inches) in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either aerating the material if it is too wet, or spraying the material with water if it is too dry. The lean clay or occasional fat clay should be compacted in the range from optimum to 3 percent wet of laboratory optimum. The native granular materials should be compacted near the laboratory optimum. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of water content. To prevent drying of the subgrade soils, placement of fill should start immediately after surface preparation and should proceed in a continuous operation until the site is brought to grade. No fill should be placed during rain or when saturation will hinder proper compaction. The finished grade surface of the compacted fill under flood wall footings should be kept in a moist condition prior to the placement of concrete. Jetting or flooding of the fill should not be permitted.

Relative compaction is the ratio of the in place dry density of the constructed fill to a maximum dry density determined by ASTM Test Designation D 1557. All levee fills should be compacted to a minimum relative compaction of 90 percent, as determined by ASTM Test Designation D 1557. All pavement aggregate base course, subbase and subgrade materials, and any fill material which is within the upper 600mm (2 feet) below finished pavement surface should be compacted to a minimum relative compaction of 95 percent. All fill material below the bottom of footings and all other structural elements should be compacted to a minimum relative compaction of 95 percent. All other fill materials should be compacted to a minimum relative compaction of 90 percent. A representative of the Geotechnical Engineer should be present to observe all grading operations during both preparation of the site and compaction of engineered fill.

The compaction and moisture content of the subgrade and each layer of levee placed should be tested by a representative of the Geotechnical Engineer in accordance with ASTM D 1556 "Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method" or ASTM D 2922 "Standard Test Method for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)", ASTM D 3017 "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)" and ASTM D 2216 "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock."

Fills placed on slopes steeper than 6:1 (horizontal to vertical) should be benched into the adjacent soils to provide a bond at the interface. Loose soils on the surface of existing slopes should be excavated as new fill is placed. Benching is expected to be necessary where new fill will be placed on existing levee slopes in order to raise the existing levee. It is recommended that horizontal benches be constructed such that the maximum height of the vertical section of the bench does not exceed 300mm (1 foot).

Where new fill is placed on the face of an existing levee, the fill should extend laterally a minimum of 600mm (2 feet) beyond the finished lines during construction. After compaction, the excess fill should be removed to the final grade and lines.

11.11 PROXIMITY TO ADJACENT EXISTING STRUCTURES

11.11.1 General

With the exception of Reaches 7A and 8, most of the alignment passes through partially or undeveloped land. In Reaches 7A and 8 there are several locations where the proposed channel improvements are in close proximity to existing structures, including the following locations (shown on the figures listed):

- Reach 7B, Station 142+50 (see Figure 120)
- Reach 8A, Station 149+25 (see Figure 121)
- Reach 8A, Station 164+50 (see Figure 122)
- Reach 8A, Station 192+50 (see Figure 123)

Each of the four figures listed above (Figures 120 through 123) includes (1) the approximate location of adjacent existing structures, (2) the approximate location of proposed channel or box culvert(s), (3) the log of nearest boring(s) showing the soil layers, (4) the groundwater level in that boring plus the highest groundwater in the adjacent piezometer (if in close proximity) and (5) the estimated maximum allowable (temporary) slope of the proposed excavation according to OSHA (1989) based on subsurface conditions. These cross sections have the same horizontal and vertical scales (1 inch equals 10 feet). Each of these cross sections will be discussed in detail in the following paragraphs.

11.11.2 Trench Section at Reach 7B Station 142+50

As shown in Figure 120, a new single barrel rectangular box culvert about 2.4m by 2.4m or 8 feet by 8 feet (width by height) is planned. For the lean clay encountered in Boring SB 7B-4 exist, we have shown a temporary side slope of 1V:3/4 H (with a dashed line). The hinge point of this excavation slope is about 1.8m (6 feet) from the building shown on the right. Assuming the building on the right is supported on spread footings, and the excavation is dewatered, shoring is not advisable. However, the right wall of the existing culvert (on the left) will be completely exposed, resulting in an unbalanced soil and water pressure on the left wall of the existing culvert. Therefore, we recommend that the District evaluate the structural stability of the existing box culvert for sliding, overturning and bearing pressure.

11.11.3 Trench Section at Reach 8A Station 149+25

As shown in Figure 121, the channel will be widened and deepened next to the existing office building (on the right) which apparently is supported on drilled piers. For the predominately clayey sand, clayey gravel and silty sand soils encountered in Borings SB 7B-7 and SB 8A-1, we have shown a temporary side slope of 1V:1½ H (with a dashed line). The hinge point of this excavation slope falls about 3.7m (12 feet) inside the footprint of the office building shown on the right. Even if this building is supported on drilled piers, shoring is recommended along the right side to prevent soil cave-in and sloughing underneath the office building.

11.11.4 Trench Section at Reach 8A Station 164+50

As shown in Figure 122, the channel will be widened and deepened next to the two existing structures: an office on the left and a house on the right. For the sandy lean clay and clayey sand soils encountered in Boring SB 8A-4, we have shown a temporary side slope of 1V:1 H (with a dashed line). The hinge point of the theoretical excavation slope is about 1.5m and 2.4m (5 feet and 8 feet) outside of the existing house and office building, respectively. Therefore, shoring at this location is inadvisable.

11.11.5 Trench Section at Reach 8A Station 192+50

As shown in Figure 123, a new double barrel rectangular box culvert is planned along Hale Avenue; each barrel is about 3.0m by 2.4m or 10 feet by 8 feet (width by height). For the fat clay and elastic silt encountered in Borings SB 8A-8 and SB 8A-8B, we have shown a temporary side slope of 1V:1 H (with a dashed line). The hinge point of the side slope falls about 1.2m (4 feet) inside the existing garage on the right. We recommend the location of the garage with respect to the property line be investigated. Depending on the Districts findings and economics, consideration might be given to (1) installing temporary shoring between the proposed box culvert and garage or (2) removal and/or replacement of the end of the garage upon completion of the box culvert. Although not shown, there may be an existing wall near the back of the garage (or other areas without a garage) that may require temporary removal and replacement. To minimize impact on the existing walls and existing garage(s) on the east side of Hale Avenue, the District also could consider moving the alignment of the proposed box culvert to the west.

11.11.6 Monitoring Program

A monitoring program is recommended to provide information to evaluate the impact of construction on the adjacent existing structures shown on Figures 120 through 123. The program should include a survey of the conditions before, during and after construction. A preconstruction survey of the existing structures should be made by a Registered Engineer in advance of the excavation. In this way, the structural and architectural condition of the structure can be documented.

Photographs and video tape showing the condition of these structures should accompany the Engineer's written description to permit post-construction comparisons. Survey monuments should be set on the existing structures; lateral movement monuments should also be installed. Periodic surveys of the monuments should be made during construction and compared to the baseline readings to measure (1) total settlement resulting from soil stress relief during excavation and to measure (2) lateral movement. The survey monuments should be installed, and baseline readings made prior to any shoring, dewatering, or excavation operations. If signs of settlement were to be detected, the situation could be corrected before the existing structures are damaged.

11.12 CONSTRUCTION DEWATERING

The groundwater levels encountered during drilling in 2003, 2004 and 2005 are in close proximity to the design channel (slightly above or below) in most Reaches. Portions of Reaches 7A and 8 may have groundwater below design channel invert. Therefore, the excavation bottoms are expected to range from relatively dry to relatively wet during the excavation phase. In the event that water is encountered, it probably could be removed by sump pit/pump procedures in combination with drainage trenches.

Depending on the season of the year, the water level could vary considerably in the Upper Llagas Creek channel. It is conceivable that some riverside maintenance road subgrade areas may be soft and wet. As discussed previously, those areas should be over-excavated and bridging material placed to a maximum depth of 600mm (2 feet); alternatively, lime stabilization treatment could be used. The water encountered could be removed by sump pit/pump procedures in combination with drainage trenches.

SECTION 12

12.1 GENERAL

Based on the field and laboratory findings and our geotechnical analyses, we believe the geotechnical condition of the foundations of new levees is generally very good. These opinions and our recommendations are discussed below in more detail.

12.2 RESULTS OF LEVEE SLOPE STABILITY EVALUATION

In Section 4 "Slope Stability Evaluation," we discussed (1) the cross sections selected for analysis, (2) method of analysis and (3) stability analysis results. In concurrence with the District, we identified a total of seventeen (17) levee sections as candidates for analysis. Based on our slope stability analyses of these 17 sections, we conclude that all but four of these sections had factors of safety that met or exceeded the minimum factor of safety required for four (4) Corps design conditions.

At Section 4 (left bank) and Section 6 (right bank) the proposed design slope is shown as 1:1. These slopes should be flattened to yield factors of safety consistent with Corps guidelines. Based on the geometry and computed factors of safety for the opposite embankment at both sections, it appears that laying these relatively steep slopes back at an inclination of 3:1 (horizontal to vertical) should yield factors of safety meeting Corps requirements. Conceivably, the slopes could be steeper than 3:1. Therefore, we recommend that the optimum slope at these two locations be determined during final design.

Furthermore, at Sections 13 and 14 in Reach 8 vertical to near vertical slopes (with gabion baskets) are shown. Due to the low factors of safety, alternative construction is recommended such as a gravity gabion retaining wall.

As previously stated, it is unlikely that the design earthquake event will coincide with a period of high water in the channel. However, all levees should be inspected following a major seismic event since deformations due to earthquake shaking (liquefaction related settlement) are anticipated. Any levee repairs and/or upgrades should be consistent with the District's levee safety program.

12.3 RESULTS OF SETTLEMENT ANALYSES

In Section 5 "Settlement Evaluation", we discussed future levee settlement. Based on new levees ranging in height from 0.3 to 1.5m (1 to 5 feet), we estimated future settlement (long term) in the range of 2 to 8mm (0.1 to 0.3 inch). This consolidation settlement is estimated to be complete in a period of 1 to 3 years. Minor overbuilding would accommodate the future settlement.

In Section 7.1.4 "Wall Settlement," consolidation settlement beneath proposed flood walls is expected to be nil.

In Section 10 "Liquefaction Potential", we estimated post-earthquake ground surface settlement in the range of 8 to 38 mm (0.3 inch to 1.5 inches) at 20 boring locations. Minor overbuilding would accommodate this future settlement.

12.4 RESULTS OF SEEPAGE ANALYSES

In Section 6 “Seepage Evaluation,” we discussed (1) method of analysis and (2) cross sections and results of analysis. In general, the levees will be constructed of well compacted lean clays with some localized fat clays. Furthermore, some of the native soils underlying the levee fills are cohesive to depths of order 5 to 10 feet. Since the cohesive soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be low. However, where mostly granular layers are encountered, seepage quantities could be moderate to large. To confirm these opinions, we performed seepage analysis on all 17 cross sections.

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee or into channel banks at the cross sections is expected to range from ¼ to 80 gallons per minute (gpm). In our opinion this estimated seepage ranges from small to large, and is conservative. This assumes a steady state condition such that seepage through the levee achieves equilibrium during the 100-year, 10-year or 5-year flood elevation. Based on our analysis, the phreatic surface for Corps Design Flood Level or Zero Freeboard extends below the ground surface in excavated sections and below the downstream toe of future levees. On this basis, remediation (such as a new cut-off trench) does not appear to be necessary.

We also recommend that any remaining armoring aggregate be completely removed prior to raising the crest of any existing levee. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope. The extent of existing armoring aggregate is unknown and should be verified in final design.

12.5 SITE PREPARATION AND GRADING

In Section 11 “Site Preparation and Earthwork,” we discussed in detail the following earthwork aspects: (1) clearing and demolition, (2) grubbing, (3) stripping, (4) removal of sediment in channel, (5) excavation, (6) subgrade preparation, (7) fill materials, including onsite materials and import materials, (8) fill placement and compaction and (9) construction dewatering. Please refer to Section 11 for details.

12.6 POST-EARTHQUAKE LEVEE INSPECTION

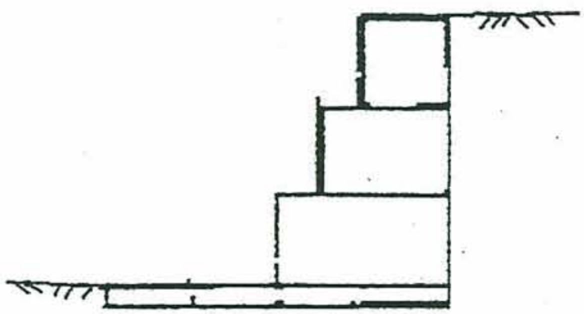
In Section 10 “Liquefaction Potential,” we presented our updated evaluation of liquefaction potential of the site and concluded that there are locations that have a relatively high potential of liquefaction during major earthquake shaking. These twenty (20) locations are also listed in Section 10. However, it is our opinion, based on limited residual strength post-liquefaction stability analyses (at 2 locations), that lateral spreading due to liquefaction is unlikely. The likely consequence that may impact the levees will be liquefaction induced settlement; we estimate such settlement to be on the order of 8 to 38mm (0.3 to 1.5 inches). Therefore, we recommend that the District inspect the levees after major earthquake events to assess the need for raising the settled crest of the levee and/or grouting underneath flood walls if gaps result from the levee settlement. In any event, the inspection and mitigation should be consistent with the District’s Levee Safety Technical Guidance Manual.

12.7 GEOTECHNICAL RECOMMENDATIONS FOR FINAL DESIGN

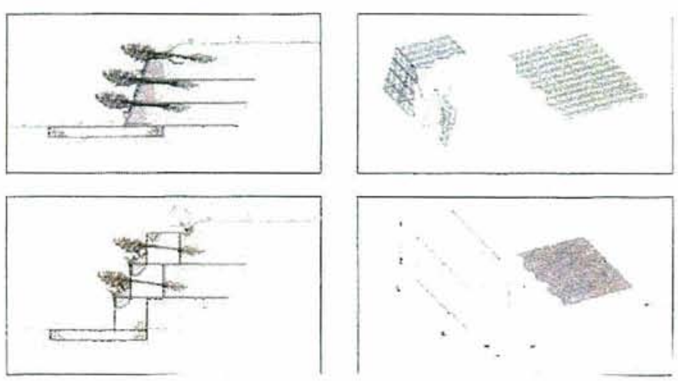
Based on the subsurface investigation, laboratory testing and analyses, we recommend the following be accomplished during final design.

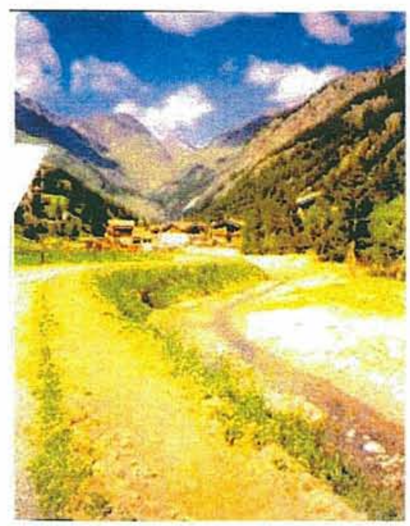
- Perform field permeability (slug) tests at the 23 new piezometers installed in 2005 and compare with the coefficient of permeability estimated for the seepage analyses in this report.
- Perform additional field exploration in the vicinity of Borings SB 6-8 and SB 6-8A, to further delineate the limits of fill in Reach 6. Concrete slabs and other miscellaneous fill were encountered and observed in this area during 2005 exploration. The limits of fill replacement should then be included on contract drawings. Conceivably, this could be done with a small backhoe.
- The locations of existing structures in close proximity to the new channel section (see Figures 120 through 123) should be surveyed in the field and reviewed during preparation of contract drawings. In particular the survey should include the trench section immediately upstream and downstream of Station 164+50 in Reach 8A.
- Continue taking water depth readings in the 23 piezometers installed for this investigation, and restart water depth readings in District piezometers installed in 2003 and 2004. Consistent with the "Final Work Plan" dated October 25, 2005, take water depth readings in all piezometers monthly, except twice a month in December, January and February. Continue readings until November 1, 2006 (changed from Final Work Plan). Provide a table or graphical plot of these water level readings in the contract documents. Assuming the project may be constructed 10 years from now, restart water depth readings 1½ years before construction commences at the monitoring frequency just described.
- When plan and profile sheets are available for flood walls and foundation drawings are available for bridge and fish barriers, a review of geotechnical design parameters should be completed to confirm their appropriateness. If drilled piers are selected for bridge foundation support, lateral load analyses should be performed when locations (horizontal and vertical) and pier diameters are known; p-y analysis may be required.
- For design of box culverts, select one of three methods for determining lateral and vertical pressures: Caltrans, AASHTO or Corps.
- Perform additional analyses of Borings SB 4-14, SB 4-15, SB 4-16, SB 6-1B, SB 6-11, SB 6-21B, SB 8A-2D, SB 8A-6B, SB 14-8, SB 14-17 and SB 14-19 for liquefaction-induced lateral spreading of granular layers.
- Specifications should require that import fill for new levees have shear strength parameters which equal or exceed $\phi'=28°$ and $c'=270$ psf.
- Due to a discrepancy in the surveyed ground surface and the estimated ground surface from a topographic map, the elevation of ground surface at Boring SB 6-1D should be resurveyed.
- The slopes at Section 4 (left bank) and Section 6 (right bank) should be flattened from 1:1 presently shown on the plans. Slope stability analyses should be performed at these two locations to determine the optimum slope inclination consistent with Corps requirements.

- In portions of Reach 8, the conceptual design includes vertical or near vertical side slopes covered with gabion baskets. Since factors of safety (FOS) are less than Corps requirements, other alternatives will need to be considered, such as:
  - Gabion gravity retaining wall. A schematic cross section and a photograph of the finished product are shown below



  - Reinforced gabion wall with a face angle of less than 70 degrees from horizontal (with horizontal straps and granular backfill). Two schematic cross sections and a photograph of the finished product are shown below.





  - MSE wall for a face angle of greater than 70 degrees from horizontal
  - Reinforced concrete retaining wall
  - Reinforced concrete U section
  - The local and global stability should be determined for the alternative that is selected for each location.
- There is a discrepancy in the documents provided regarding the right and left directions (when viewing channel cross sections), as to whether the directions are determined looking

upstream or downstream. To avoid confusion during final design, the Corps/District should select one view (either looking upstream or looking downstream, but not both).

- The Corps/District should perform a study of erosion control and slope protection measures and consider the effect of erosion on the slope stability calculations.
- At present there is a discrepancy of centerline stationing in the available reports and plans since two different systems have been used: "Corps Hydraulic Stationing" and "Corps Design Stationing". To avoid confusion during final design, the Corps/District should select one system.

SECTION 13

The conclusions and recommendations contained in this report are based on exploratory borings drilled for this study, as well as our review of available subsurface information, and our local experience and engineering judgment. Furthermore, slope stability analyses have been limited to seventeen (17) critical cross sections.

Surveyed ground elevations and coordinates of borings were provided by the District.

An investigation for subsurface environmental contamination was beyond the scope of services.

A study of erosion potential, erosion control and slope protection measures was beyond the scope of services.

The recommendations presented in this report were developed with the standard of care commonly used in this profession. No other warranties are included, either express or implied, as to the professional advice presented in this report.

SECTION 14

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Aerial Photographs

Black and white

Date	Approx. Scale	Project	Roll/Frame No.	Source
10/20/39	1:20,000	CIV	293/16 – 9	Nat'l Archives
10/20/30	1:20,000	CIV	294/57 – 61	Nat'l Archives

TABLES

Table 1 - Calculated PGAs for Site No. 4-A

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	3.5	0.48	0.53	0.47	0.49	0.74	0.89	0.70	0.77
San Andreas - Peninsula Segment	7.9	SS	41.8	0.15	0.19	0.17	0.17	0.22	0.32	0.26	0.27
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.4	0.28	0.40	0.34	0.34	0.44	0.66	0.50	0.53
Hayward SE Extension	6.5	SS	29.5	0.11	0.12	0.11	0.11	0.19	0.20	0.18	0.19
Hayward	6.9	SS	51.6	0.08	0.10	0.08	0.08	0.13	0.16	0.11	0.13
Sargent	6.8	OR	11.2	0.30	0.33	0.36	0.33	0.47	0.56	0.56	0.53
Monte Vista - Shannon	6.8	R	30.7	0.15	0.16	0.16	0.16	0.24	0.27	0.25	0.25
San Gregorio	7.5	SS	58.5	0.10	0.12	0.10	0.10	0.15	0.20	0.15	0.16
Zayante - Vergeles	6.8	R	20.1	0.21	0.22	0.24	0.23	0.34	0.38	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 2 - Calculated PGAs for Site No. 4-B

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	4.1	0.46	0.51	0.45	0.47	0.71	0.85	0.67	0.75
San Andreas - Peninsula Segment	7.9	SS	40.7	0.15	0.19	0.18	0.17	0.23	0.32	0.26	0.27
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.1	0.29	0.40	0.34	0.34	0.44	0.67	0.51	0.54
Hayward SE Extension	6.5	SS	28.6	0.12	0.12	0.11	0.12	0.19	0.20	0.18	0.19
Hayward	6.9	SS	50.7	0.08	0.10	0.08	0.09	0.13	0.16	0.12	0.14
Sargent	6.8	OR	10.9	0.30	0.34	0.37	0.34	0.48	0.57	0.57	0.54
Monte Vista - Shannon	6.8	R	29.6	0.16	0.17	0.17	0.16	0.25	0.28	0.26	0.26
San Gregorio	7.5	SS	57.7	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	19.7	0.22	0.23	0.24	0.23	0.34	0.38	0.37	0.37

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 3 - Calculated PGAs for Site No. 4-C

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	4.6	0.45	0.49	0.44	0.46	0.69	0.82	0.65	0.72
San Andreas - Peninsula Segment	7.9	SS	39.5	0.15	0.20	0.18	0.18	0.23	0.33	0.27	0.28
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.0	0.29	0.40	0.34	0.34	0.44	0.68	0.51	0.54
Hayward SE Extension	6.5	SS	27.3	0.12	0.12	0.12	0.12	0.20	0.21	0.19	0.20
Hayward	6.9	SS	49.4	0.08	0.10	0.08	0.09	0.13	0.17	0.12	0.14
Sargent	6.8	OR	10.7	0.31	0.34	0.37	0.34	0.48	0.57	0.57	0.54
Monte Vista - Shannon	6.8	R	28.3	0.16	0.17	0.18	0.17	0.25	0.29	0.27	0.27
San Gregorio	7.5	SS	57.1	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	19.5	0.22	0.23	0.24	0.23	0.35	0.38	0.38	0.37

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 4 - Calculated PGAs for Site No. 6-D

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	5.3	0.42	0.47	0.42	0.44	0.65	0.79	0.63	0.69
San Andreas - Peninsula Segment	7.9	SS	38.5	0.15	0.20	0.19	0.18	0.24	0.34	0.28	0.28
San Andreas - Sta Cruz Mtns Segment	7.9	SS	14.6	0.29	0.41	0.34	0.35	0.45	0.69	0.51	0.55
Hayward SE Extension	6.5	SS	26.6	0.12	0.13	0.12	0.12	0.20	0.21	0.20	0.21
Hayward	6.9	SS	48.6	0.09	0.10	0.08	0.09	0.13	0.17	0.12	0.14
Sargent	6.8	OR	10.3	0.31	0.35	0.38	0.35	0.50	0.59	0.59	0.56
Monte Vista - Shannon	6.8	R	27.4	0.17	0.18	0.18	0.17	0.26	0.30	0.28	0.28
San Gregorio	7.5	SS	56.4	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	19.0	0.22	0.23	0.25	0.24	0.35	0.39	0.39	0.38

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 5 - Calculated PGAs for Site No. 6-E

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	5.2	0.43	0.47	0.42	0.44	0.66	0.79	0.63	0.69
San Andreas - Peninsula Segment	7.9	SS	37.2	0.16	0.21	0.19	0.19	0.24	0.35	0.28	0.29
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.0	0.29	0.40	0.34	0.34	0.44	0.68	0.51	0.54
Hayward SE Extension	6.5	SS	25.0	0.13	0.13	0.13	0.13	0.22	0.22	0.21	0.22
Hayward	6.9	SS	47.1	0.09	0.10	0.08	0.09	0.14	0.17	0.13	0.15
Sargent	6.8	OR	10.4	0.31	0.35	0.38	0.35	0.49	0.58	0.58	0.55
Monte Vista - Shannon	6.8	R	25.9	0.17	0.19	0.19	0.18	0.27	0.31	0.29	0.29
San Gregorio	7.5	SS	55.9	0.10	0.12	0.10	0.11	0.15	0.21	0.15	0.17
Zayante - Vergeles	6.8	R	19.3	0.22	0.23	0.25	0.23	0.35	0.39	0.38	0.37

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 6 - Calculated PGAs for Site No. 6-F

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	5.2	0.43	0.47	0.42	0.44	0.66	0.79	0.63	0.69
San Andreas - Peninsula Segment	7.9	SS	36.3	0.16	0.21	0.19	0.19	0.25	0.35	0.29	0.30
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.3	0.29	0.40	0.34	0.34	0.44	0.67	0.50	0.54
Hayward SE Extension	6.5	SS	23.6	0.14	0.14	0.14	0.14	0.23	0.23	0.22	0.23
Hayward	6.9	SS	45.6	0.09	0.10	0.09	0.09	0.14	0.18	0.13	0.15
Sargent	6.8	OR	109.0	0.04	0.06	0.03	0.05	0.07	0.10	0.05	0.07
Monte Vista - Shannon	6.8	R	24.7	0.18	0.19	0.20	0.19	0.29	0.32	0.31	0.30
San Gregorio	7.5	SS	56.0	0.10	0.12	0.10	0.11	0.15	0.21	0.15	0.17
Zayante - Vergeles	6.8	R	20.0	0.21	0.22	0.24	0.23	0.34	0.38	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 7 - Calculated PGAs for Site No. 6-G

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	6.0	0.40	0.44	0.41	0.42	0.62	0.75	0.60	0.66
San Andreas - Peninsula Segment	7.9	SS	35.0	0.16	0.22	0.20	0.19	0.25	0.36	0.30	0.31
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.0	0.29	0.40	0.34	0.34	0.44	0.68	0.51	0.54
Hayward SE Extension	6.5	SS	22.4	0.14	0.14	0.14	0.14	0.24	0.24	0.23	0.24
Hayward	6.9	SS	44.4	0.09	0.11	0.09	0.10	0.14	0.18	0.14	0.15
Sargent	6.8	OR	10.9	0.30	0.34	0.37	0.34	0.48	0.57	0.57	0.54
Monte Vista - Shannon	6.8	R	23.3	0.19	0.20	0.21	0.20	0.30	0.34	0.32	0.32
San Gregorio	7.5	SS	55.4	0.10	0.12	0.10	0.11	0.15	0.21	0.16	0.17
Zayante - Vergeles	6.8	R	20.1	0.21	0.22	0.24	0.23	0.34	0.38	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 8 - Calculated PGAs for Site No. 7-H

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	7.0	0.38	0.41	0.38	0.39	0.58	0.70	0.57	0.62
San Andreas - Peninsula Segment	7.9	SS	33.6	0.17	0.22	0.21	0.20	0.26	0.38	0.31	0.32
San Andreas - Sta Cruz Mtns Segment	7.9	SS	14.4	0.30	0.41	0.35	0.35	0.46	0.70	0.52	0.56
Hayward SE Extension	6.5	SS	21.2	0.15	0.15	0.15	0.15	0.25	0.25	0.25	0.25
Hayward	6.9	SS	43.2	0.09	0.11	0.09	0.10	0.15	0.18	0.14	0.16
Sargent	6.8	OR	10.5	0.31	0.35	0.38	0.34	0.49	0.58	0.58	0.55
Monte Vista - Shannon	6.8	R	21.9	0.20	0.21	0.22	0.21	0.31	0.35	0.34	0.34
San Gregorio	7.5	SS	54.4	0.10	0.13	0.11	0.11	0.16	0.21	0.16	0.17
Zayante - Vergeles	6.8	R	20.0	0.21	0.22	0.24	0.23	0.34	0.38	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 9 - Calculated PGAs for Site No. 7-I

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	6.9	0.38	0.42	0.39	0.39	0.58	0.70	0.58	0.62
San Andreas - Peninsula Segment	7.9	SS	32.5	0.17	0.23	0.21	0.21	0.27	0.39	0.32	0.32
San Andreas - Sta Cruz Mtns Segment	7.9	SS	14.5	0.30	0.41	0.35	0.35	0.45	0.69	0.52	0.55
Hayward SE Extension	6.5	SS	19.9	0.16	0.16	0.16	0.16	0.26	0.27	0.26	0.26
Hayward	6.9	SS	41.9	0.10	0.11	0.10	0.10	0.15	0.19	0.15	0.16
Sargent	6.8	OR	11.1	0.30	0.33	0.36	0.33	0.47	0.56	0.56	0.53
Monte Vista - Shannon	6.8	R	20.5	0.21	0.22	0.23	0.22	0.33	0.37	0.36	0.35
San Gregorio	7.5	SS	54.2	0.10	0.13	0.11	0.11	0.16	0.21	0.16	0.18
Zayante - Vergeles	6.8	R	20.1	0.21	0.22	0.24	0.23	0.34	0.38	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 10 - Calculated PGAs for Site No. 8-J

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	6.6	0.39	0.43	0.39	0.40	0.59	0.72	0.59	0.63
San Andreas - Peninsula Segment	7.9	SS	31.6	0.18	0.23	0.22	0.21	0.27	0.39	0.32	0.33
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.2	0.29	0.40	0.34	0.34	0.44	0.67	0.50	0.54
Hayward SE Extension	6.5	SS	18.3	0.17	0.17	0.17	0.17	0.28	0.28	0.28	0.28
Hayward	6.9	SS	40.2	0.10	0.12	0.10	0.11	0.16	0.19	0.15	0.17
Sargent	6.8	OR	12.0	0.28	0.32	0.35	0.32	0.45	0.53	0.53	0.51
Monte Vista - Shannon	6.8	R	19.2	0.22	0.23	0.25	0.23	0.35	0.39	0.38	0.37
San Gregorio	7.5	SS	54.1	0.10	0.13	0.11	0.11	0.16	0.21	0.16	0.18
Zayante - Vergeles	6.8	R	20.8	0.21	0.22	0.23	0.22	0.33	0.37	0.36	0.35

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 11 - Calculated PGAs for Site No. 8-K

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	6.8	0.38	0.42	0.39	0.40	0.59	0.71	0.58	0.62
San Andreas - Peninsula Segment	7.9	SS	30.2	0.18	0.24	0.22	0.22	0.28	0.41	0.33	0.34
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.1	0.29	0.40	0.34	0.34	0.44	0.67	0.51	0.54
Hayward SE Extension	6.5	SS	16.9	0.18	0.18	0.19	0.18	0.30	0.30	0.30	0.30
Hayward	6.9	SS	38.7	0.10	0.12	0.10	0.11	0.16	0.20	0.16	0.17
Sargent	6.8	OR	11.6	0.29	0.32	0.35	0.32	0.46	0.55	0.55	0.52
Monte Vista - Shannon	6.8	R	17.7	0.24	0.24	0.26	0.25	0.37	0.41	0.41	0.40
San Gregorio	7.5	SS	53.5	0.10	0.13	0.11	0.11	0.16	0.21	0.16	0.18
Zayante - Vergeles	6.8	R	20.9	0.21	0.22	0.23	0.22	0.33	0.36	0.36	0.35

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 12 - Calculated PGAs for Site No. 14-L

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	4.2	0.46	0.50	0.45	0.47	0.70	0.85	0.67	0.74
San Andreas - Peninsula Segment	7.9	SS	38.5	0.15	0.20	0.19	0.18	0.24	0.34	0.28	0.28
San Andreas - Sta Cruz Mtns Segment	7.9	SS	15.8	0.28	0.39	0.33	0.33	0.43	0.65	0.49	0.53
Hayward SE Extension	6.5	SS	25.7	0.13	0.13	0.13	0.13	0.21	0.22	0.20	0.21
Hayward	6.9	SS	47.8	0.09	0.10	0.08	0.09	0.14	0.17	0.13	0.14
Sargent	6.8	OR	11.4	0.29	0.33	0.36	0.33	0.46	0.55	0.55	0.52
Monte Vista - Shannon	6.8	R	27.0	0.17	0.18	0.18	0.18	0.27	0.30	0.28	0.28
San Gregorio	7.5	SS	67.2	0.09	0.11	0.08	0.09	0.13	0.18	0.13	0.15
Zayante - Vergeles	6.8	R	20.2	0.21	0.22	0.24	0.22	0.34	0.37	0.37	0.36

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 13 - Calculated PGAs for Site No. 14-M

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	4.2	0.46	0.50	0.45	0.47	0.70	0.85	0.67	0.74
San Andreas - Peninsula Segment	7.9	SS	37.4	0.16	0.21	0.19	0.18	0.24	0.35	0.28	0.29
San Andreas - Sta Cruz Mtns Segment	7.9	SS	16.2	0.28	0.38	0.33	0.33	0.42	0.64	0.49	0.52
Hayward SE Extension	6.5	SS	24.1	0.14	0.14	0.13	0.14	0.22	0.23	0.22	0.22
Hayward	6.9	SS	46.3	0.09	0.10	0.09	0.09	0.14	0.17	0.13	0.15
Sargent	6.8	OR	11.7	0.29	0.32	0.35	0.32	0.46	0.54	0.54	0.51
Monte Vista - Shannon	6.8	R	25.7	0.17	0.19	0.19	0.18	0.28	0.31	0.30	0.30
San Gregorio	7.5	SS	57.0	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	20.8	0.21	0.22	0.23	0.22	0.33	0.37	0.36	0.35

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 14 - Calculated PGAs for Site No. 14-N

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	4.7	0.44	0.49	0.44	0.46	0.68	0.82	0.65	0.72
San Andreas - Peninsula Segment	7.9	SS	36.3	0.16	0.21	0.19	0.19	0.25	0.35	0.29	0.30
San Andreas - Sta Cruz Mtns Segment	7.9	SS	16.2	0.28	0.38	0.33	0.33	0.42	0.64	0.49	0.52
Hayward SE Extension	6.5	SS	22.7	0.14	0.14	0.14	0.14	0.23	0.24	0.23	0.24
Hayward	6.9	SS	448.0	0.01	0.02	0.00	0.01	0.02	0.03	0.00	0.02
Sargent	6.8	OR	11.9	0.29	0.32	0.35	0.32	0.45	0.54	0.54	0.51
Monte Vista - Shannon	6.8	R	24.3	0.18	0.19	0.20	0.19	0.29	0.33	0.31	0.31
San Gregorio	7.5	SS	56.7	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	21.3	0.20	0.21	0.23	0.21	0.32	0.36	0.35	0.34

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 15 - Calculated PGAs for Site No. 14-O

FAULT	Maximum Magnitude (M_w)	Sense of Faulting[1]	Rupture Distance (km)	Calculated PGA (g), Median				Calculated PGA (g), 84th			
				Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average	Abrahamson & Silva (97)	Boore et. al., (97)	Sadigh et. al., (97)	Weighted Average
Calaveras	7.1	SS	5.1	0.43	0.47	0.43	0.44	0.66	0.80	0.64	0.70
San Andreas - Peninsula Segment	7.9	SS	35.5	0.16	0.21	0.20	0.19	0.25	0.36	0.30	0.30
San Andreas - Sta Cruz Mtns Segment	7.9	SS	16.2	0.28	0.38	0.33	0.33	0.42	0.64	0.49	0.52
Hayward SE Extension	6.5	SS	21.8	0.15	0.15	0.15	0.15	0.24	0.25	0.24	0.24
Hayward	6.9	SS	43.9	0.09	0.11	0.09	0.10	0.15	0.18	0.14	0.15
Sargent	6.8	OR	12.1	0.28	0.32	0.35	0.31	0.45	0.53	0.53	0.50
Monte Vista - Shannon	6.8	R	23.5	0.19	0.20	0.21	0.20	0.30	0.33	0.32	0.32
San Gregorio	7.5	SS	56.4	0.10	0.12	0.10	0.11	0.15	0.20	0.15	0.17
Zayante - Vergeles	6.8	R	21.3	0.20	0.21	0.23	0.21	0.32	0.36	0.35	0.34

[1] SS - Strike-Slip; R - Reverse; OR - Oblique-Reverse

Table 16A
Observation Well Readings
Reach 4

Date / Well Number	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	06-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
OW_4-6	222.53	222.43	222.33	223.23	222.03	232.63	230.93	230.73	231.63
OW_4-11	214.90	214.70	214.60	215.20	214.50	224.60	223.00	222.70	223.30
OW_4-13	208.60	207.80	207.80	207.20	207.20	221.40	219.60	219.30	220.00
OW_4-18	204.78	204.78	204.78	204.78	204.78	213.98	212.38	211.88	213.18

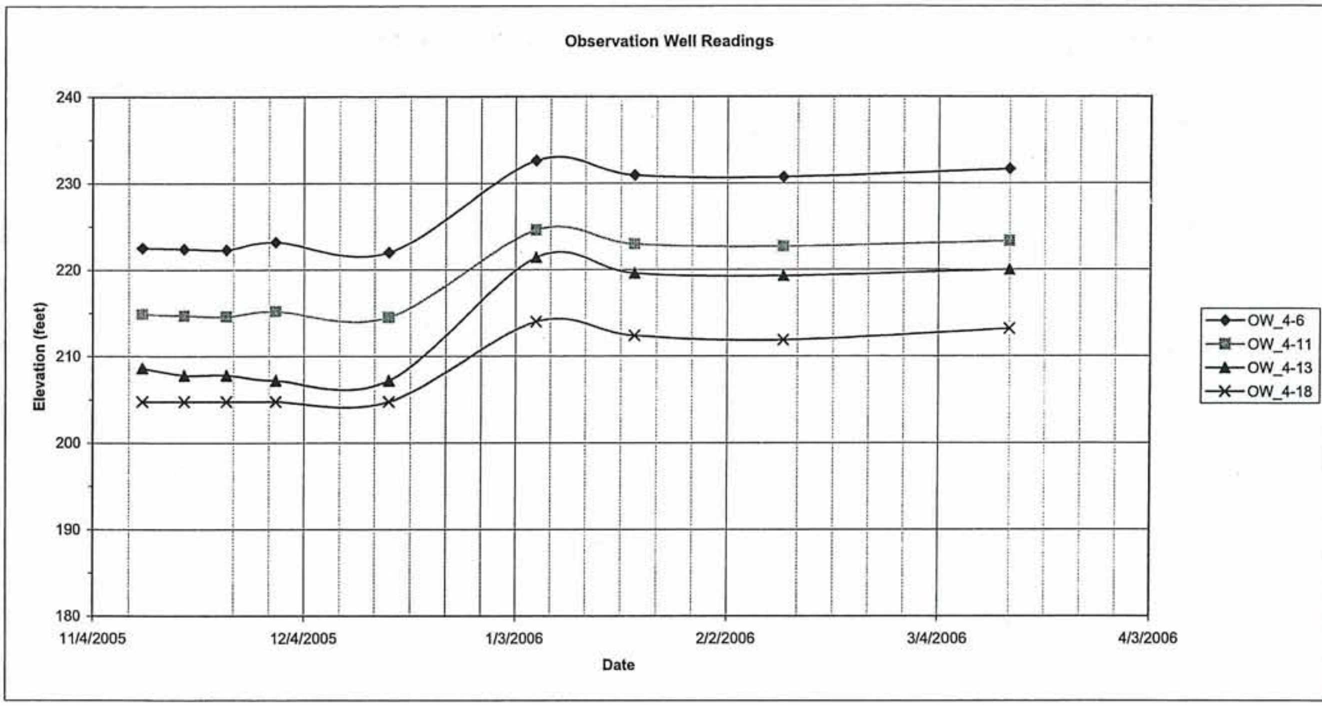


Table 16B
Observation Well Readings
Reach 5

Date	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	6-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
Well Number									
OW_5-3	230.54	230.54	230.44	232.84	232.44	243.44	241.24	240.74	241.34

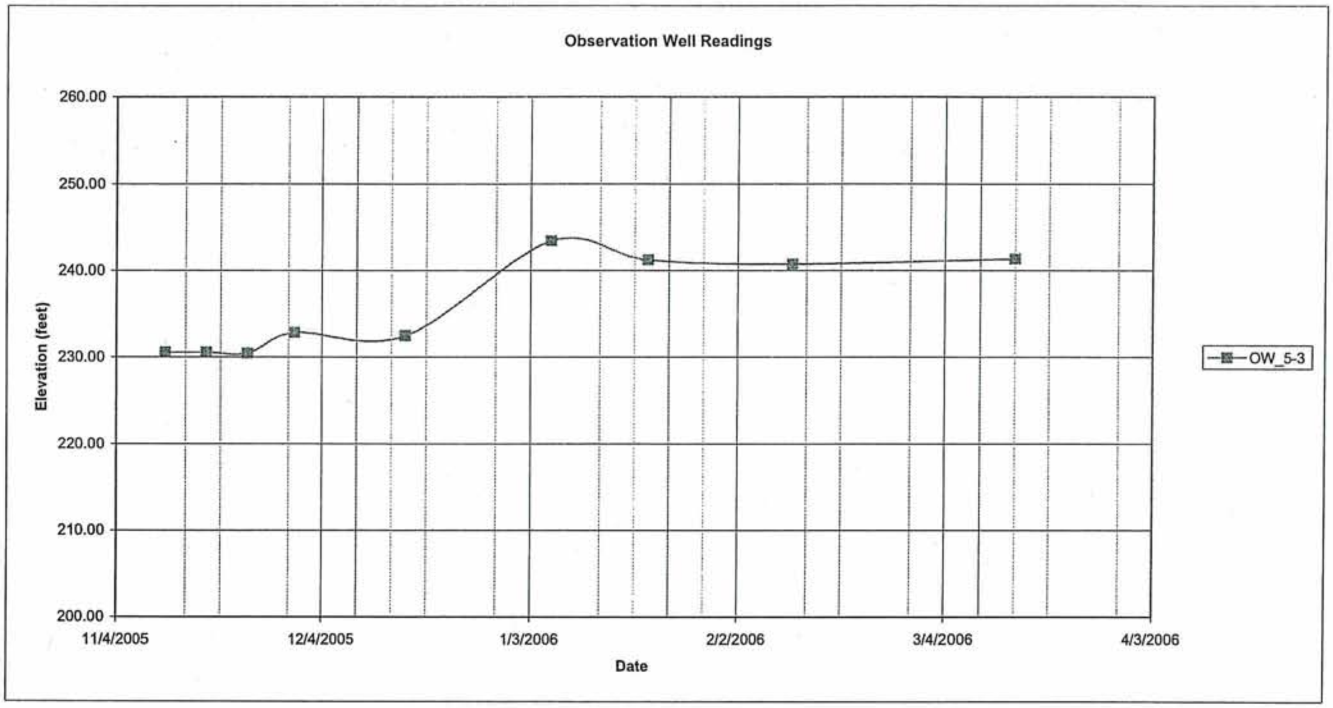


Table 16C
Observation Well Readings
Reach 6

Date / Well Number	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	06-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
OW_6-7	288.79	288.89	288.79	289.79	288.69	292.59	289.79	289.39	289.99
OW_6-12	270.41	270.61	270.51	273.51	270.51	282.51	278.81	278.71	279.01
OW_6-17	265.67	266.87	266.87	270.57	265.67	273.57	270.57	270.77	272.47
OW_6-25	250.25	250.65	250.45	252.05	250.45	254.25	251.85	251.55	251.15
OW_6-27								247.88	248.28

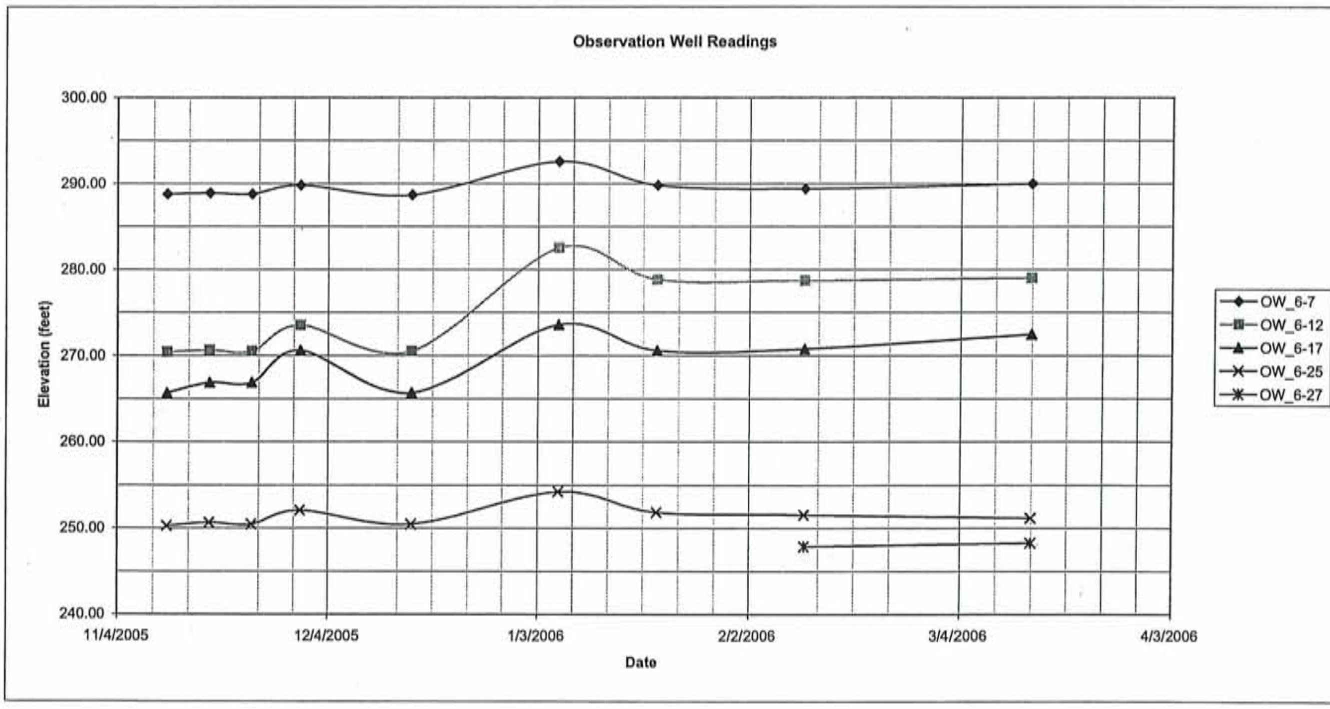


Table 16D
Observation Well Readings
Reach 7A and 7B

Date / Well Number	19-Jul-04	15-Sep-04	27-Oct-04	10-Dec-04	14-Jan-05	9-Mar-05
PIEZOMETER 7A-9	296.97	294.67	294.37	294.47		
PIEZOMETER 7A-10	297.30	292.90	292.10	292.20	309.30	310.80
PIEZOMETER 7A-11	298.00	293.30	293.00	294.00	304.10	309.70
PIEZOMETER 7A-H4	299.50	299.20	298.50	298.80	304.90	307.00

Date / Well Number	16-Jun-03	15-Jul-03	17-Sep-03	15-Oct-03	17-Dec-03	9-Jan-04	18-Feb-04	11-Mar-04	15-Apr-04	27-May-04	19-Jul-04	15-Sep-04	27-Oct-04	10-Dec-04	14-Jan-05	9-Mar-05
PIEZOMETER B-2	331.33	330.08	328.28	327.28	329.78	331.78	332.18	332.38	331.38	330.48	328.28	325.88		329.38	332.18	332.98
PIEZOMETER B-3	331.52	330.44	328.42	327.52	329.72	332.52	333.12	333.72	332.52	330.92	328.72	326.02	327.12	328.22	332.72	334.42
PIEZOMETER B-5	332.73	330.33	328.33	327.43	329.73	332.73	333.23	334.13	332.73	330.93	328.63	325.93	326.83	326.12	331.22	333.12
PIEZOMETER B-7	329.68	328.08	325.78	324.68	326.58	329.68	332.38	333.68	332.08	329.38	326.18	321.08	322.48	323.82	330.72	333.52

Date / Well Number	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	06-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
OW_7A-2	310.02	310.02	310.02	310.02	310.02	310.02	310.02	313.42	316.42
OW_7B-4B	323.58	324.08	324.18	325.28	324.88	327.68	327.98	328.08	328.48

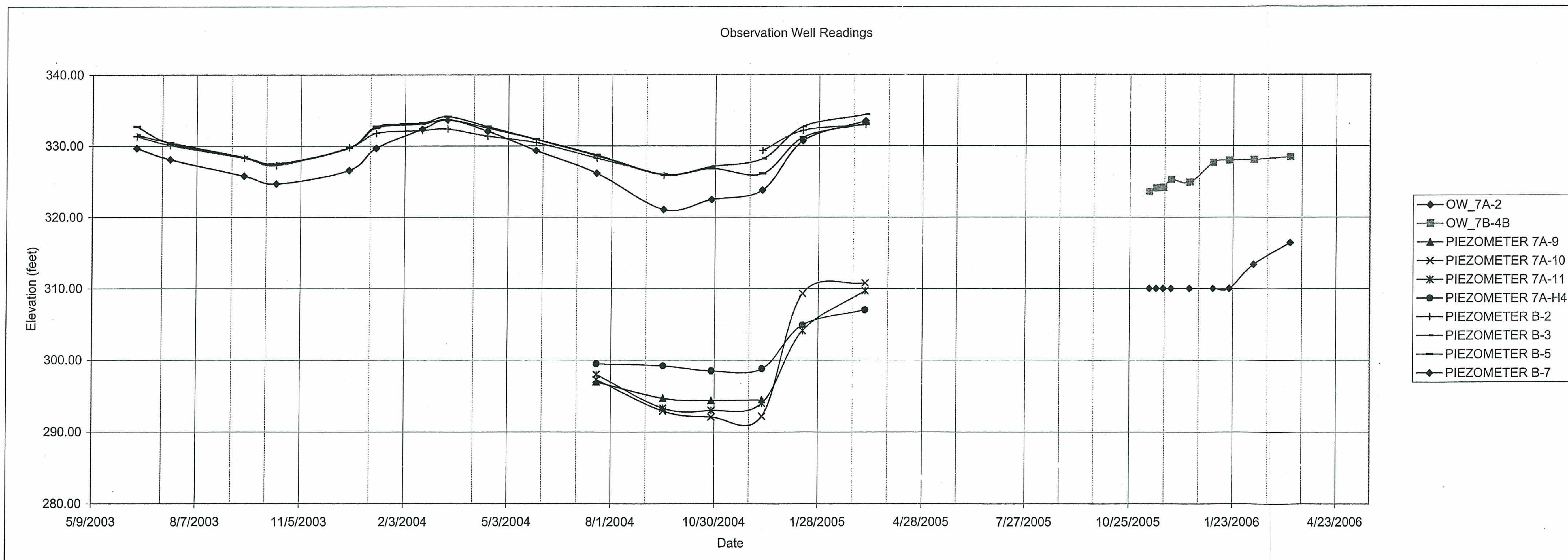


Table 16E
Observation Well Readings
Reach 8

Date / Well Number	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	06-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
OW_8A-1B	323.53	324.63	324.43	326.83	326.93	333.03	332.03	331.83	334.13
OW_8A-2C	323.26	323.56	323.76	325.26	325.96	334.76	334.86	334.66	334.56
OW_8A-3B	324.27	326.67	326.27	330.87	326.47	335.87	335.97	336.07	336.07
OW_8A-4	326.57	327.27	326.27	330.47	326.27	335.77	338.17	338.27	338.67
OW_8A-5	330.89	331.39	331.09	332.19	331.89	336.59	337.09	337.49	338.19

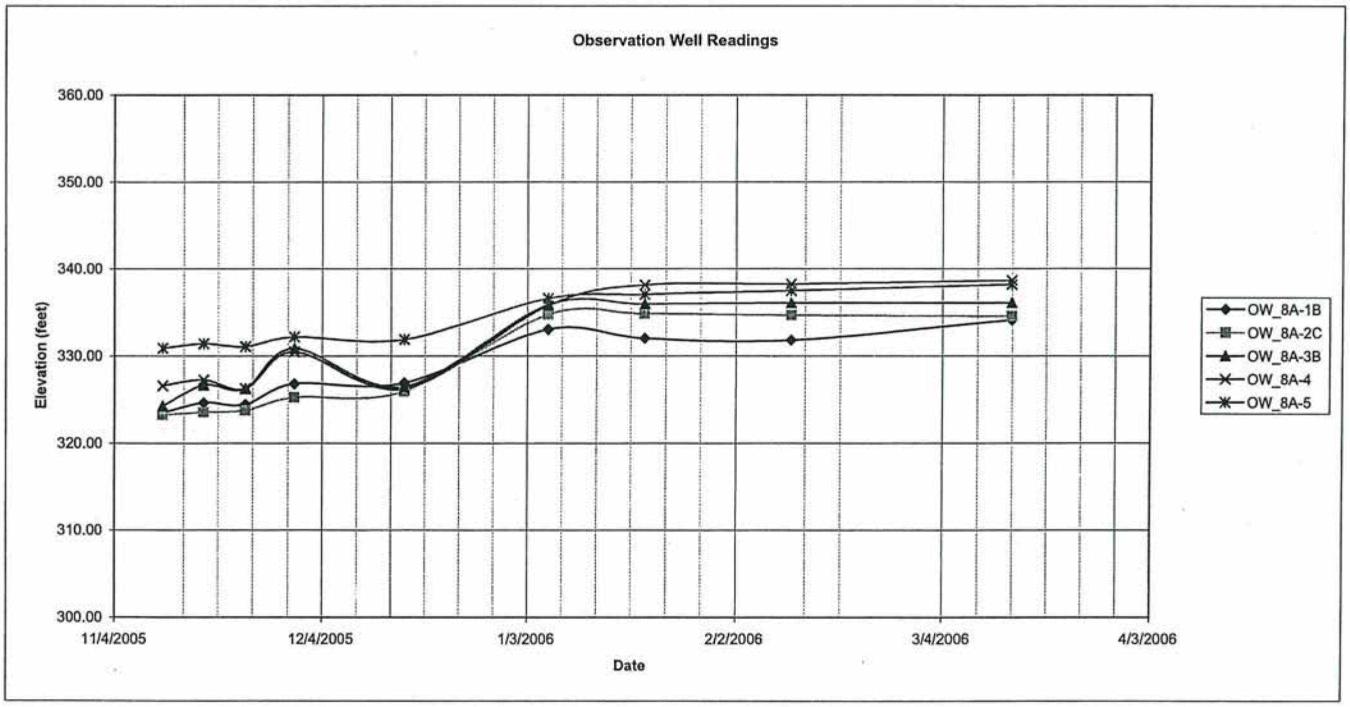


Table 16F
Observation Well Readings
Reach 14

Date / Well Number	11-Nov-05	17-Nov-05	23-Nov-05	30-Nov-05	16-Dec-05	06-Jan-06	20-Jan-06	10-Feb-06	14-Mar-06
OW_14-3	287.42	287.42	287.42	290.72	287.42	297.82	298.12	297.52	298.32
OW_14-10	271.00	271.00	271.00	282.10	271.00	283.50	283.50	282.90	284.00
OW_14-16	254.13	253.93	253.83	253.73	258.13	266.33	265.53	265.03	268.33
OW_14-22	247.51	247.51	247.61	250.01	248.41	253.01	252.61	249.61	253.61
OW_14-26	232.38	232.38	232.38	232.38	232.38	242.38	241.68	241.48	243.38
OW_14-27	230.34	230.34	230.34	230.34	230.34	241.24	239.94	239.74	240.24

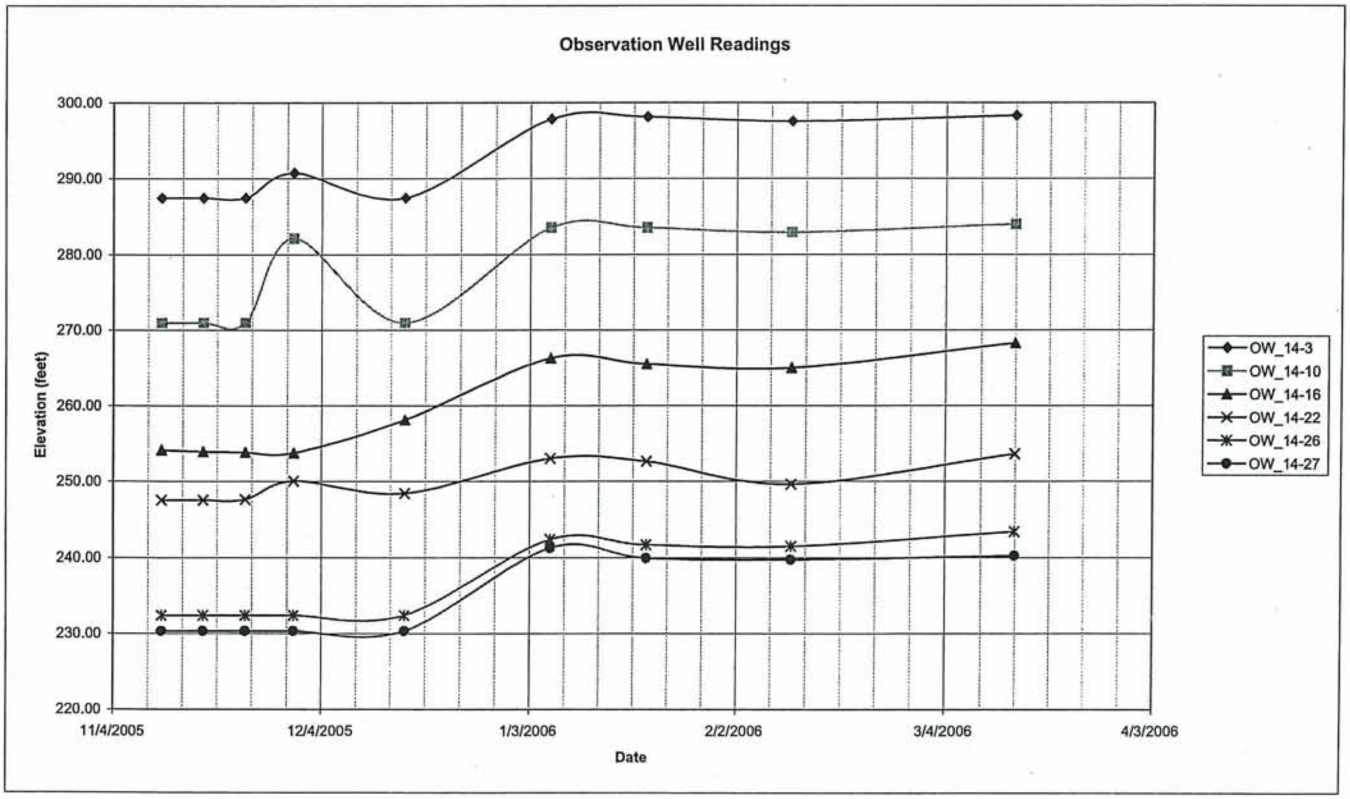


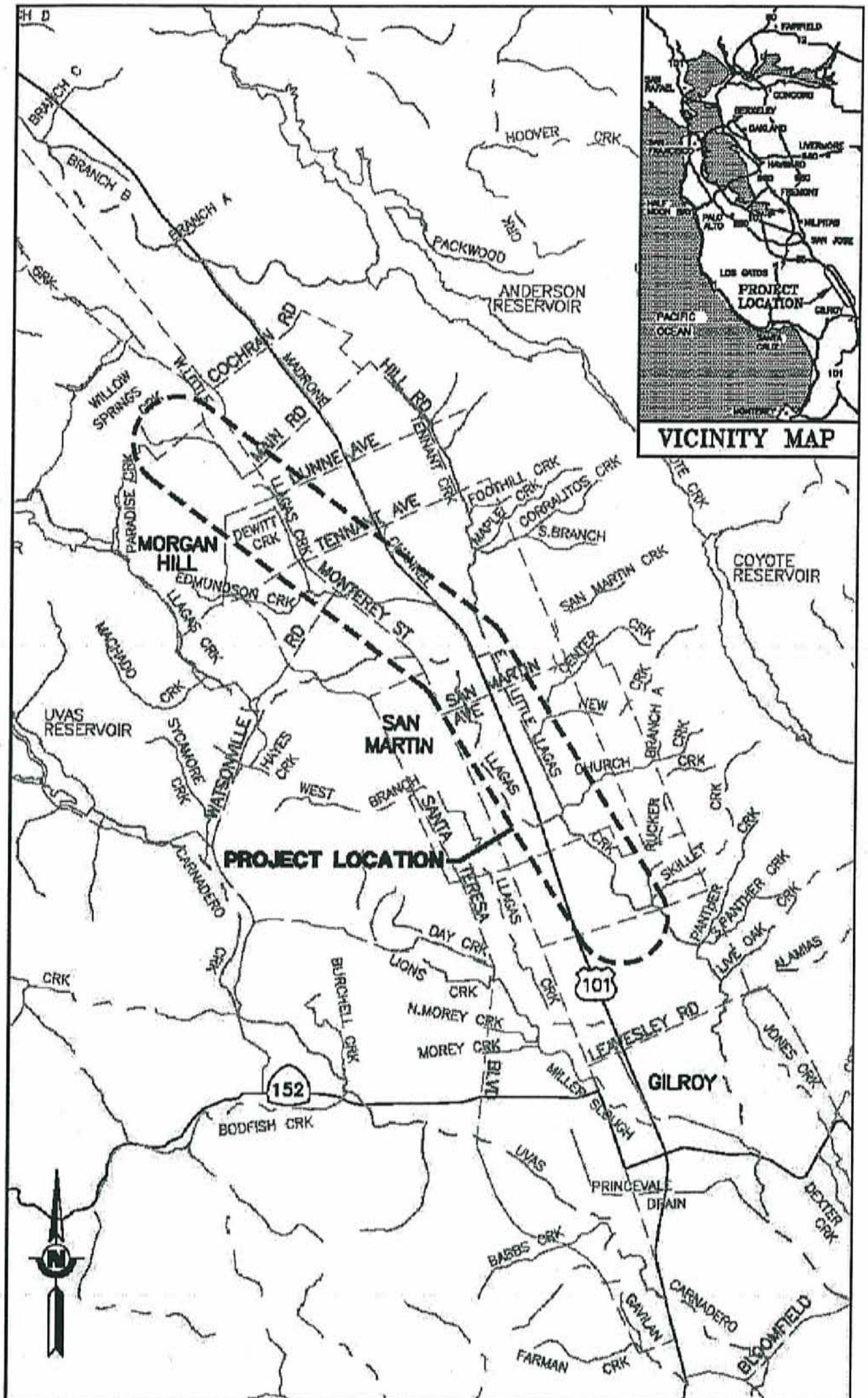
Table 17 – Recommended Structural Pavement Section

R-Value	Asphalt Concrete Pavement Structural Design Thicknesses (mm)														
	TI = 4.0			TI = 4.5			TI = 5.0			TI = 5.5			TI = 6.0		
	AC	AB	AS	AC	AB	AS	AC	AB	AS	AC	AB	AS	AC	AB	AS
5	60	105	105	60	120	135	60	135	165	75	135	195	75	165	210
5	60	210	0	60	240	0	60	285	0	75	315	0	90	315	0
8	60	105	105*	60	120	120	60	135	150	75	135	180	75	165	195
8	60	195	0	60	240	0	60	270	0	75	300	0	90	300	0
10	60	105	105*	60	120	120	60	135	150	75	135	165	75	165	180
10	60	195	0	60	225	0	60	270	0	75	285	0	90	300	0
11	60	105	105*	60	120	105	60	135	135	75	135	165	75	165	165
11	60	180	0	60	210	0	60	255	0	75	285	0	75	315	0
14	60	105	105*	60	120	105*	60	135	120	75	135	150	75	165	150
14	60	165	0	60	210	0	60	255	0	75	270	0	75	300	0
15	60	105	105*	60	120	105*	60	135	120	75	135	135	75	165	150
15	60	165	0	60	210	0	60	240	0	75	255	0	75	300	0
19	60	105	105*	60	120	105*	60	135	105*	75	135	120	75	165	120
19	60	150	0	60	195	0	60	225	0	75	240	0	75	285	0

Note: *The thickness of aggregate subbase layer is governed by a minimum thickness requirement per Caltrans' guideline. Alternative pavement section without aggregate subbase layer is recommended.

AC = Asphalt Concrete
AB = Class 2 Aggregate Base, R = 78
AS = Class 4 Aggregate Subbase, R = 50

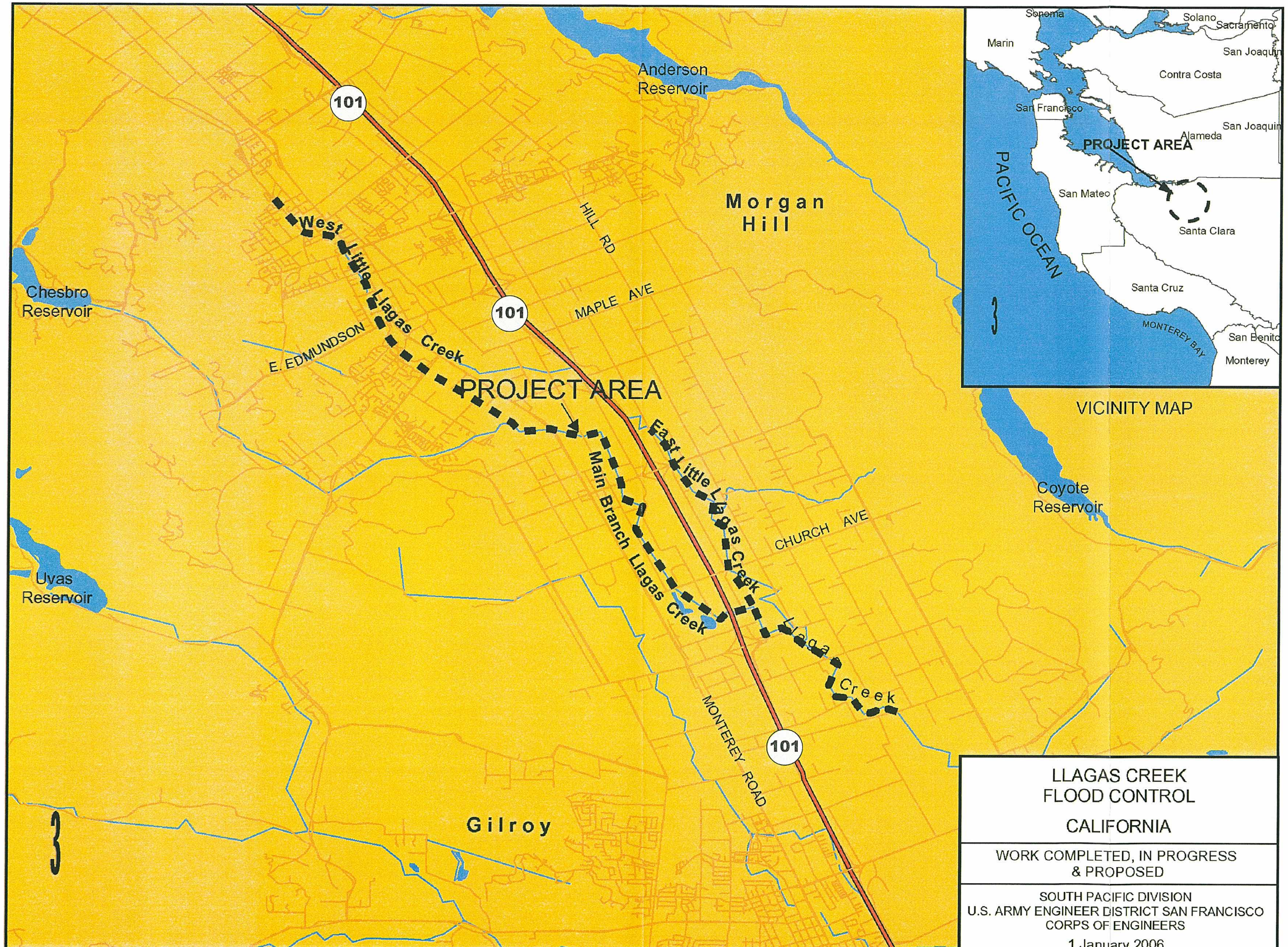
FIGURES



LOCATION MAP

URS	**Upper Llagas Creek Flood Protection Project** Santa Clara County, CA	**LOCATION MAP**	NOT TO SCALE Figure **1** 5/19/06

Q:\CADD2\2006\28649692\GE-692008.dwg



LLAGAS CREEK
FLOOD CONTROL
CALIFORNIA

WORK COMPLETED, IN PROGRESS & PROPOSED

SOUTH PACIFIC DIVISION
U.S. ARMY ENGINEER DISTRICT SAN FRANCISCO
CORPS OF ENGINEERS
1 January 2006

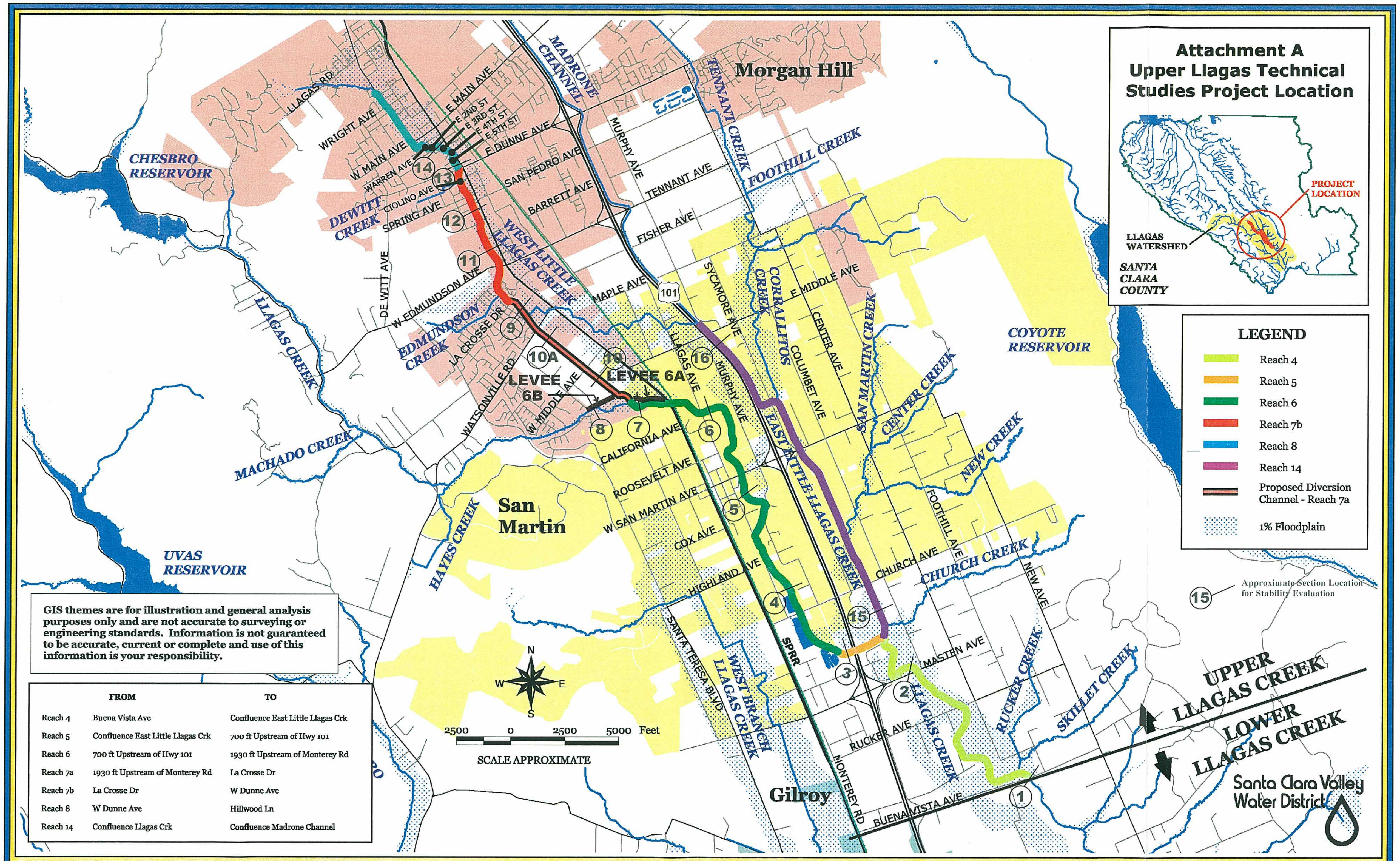
O:\CADD2\2006\28649692\GE-692006.dwg

URS

Upper Llagas Creek Flood Protection Project
Santa Clara County, CA

SITE MAP

NOT TO SCALE
Figure
2
5/19/06



	FROM	TO
Reach 4	Buena Vista Ave	Confluence East Little Llagas Crk
Reach 5	Confluence East Little Llagas Crk	700 ft Upstream of Hwy 101
Reach 6	700 ft Upstream of Hwy 101	1930 ft Upstream of Monterey Rd
Reach 7a	1930 ft Upstream of Monterey Rd	La Crosse Dr
Reach 7b	La Crosse Dr	W Dunne Ave
Reach 8	W Dunne Ave	Hillwood Ln
Reach 14	Confluence Llagas Crk	Confluence Madrone Channel

26174052\2005_072\llagas_project_location.apr 06/02/2005

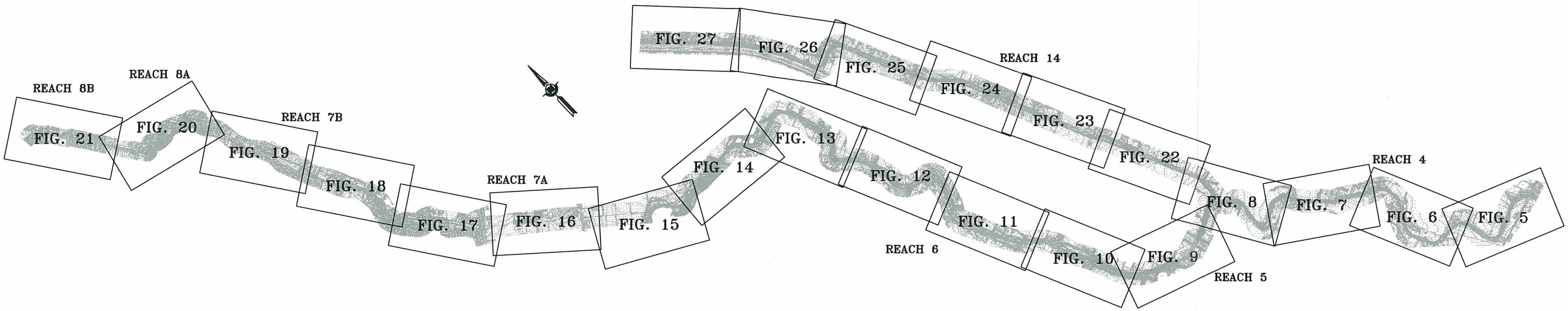
O:\CADD2\2005\28649692_ULlagasFPP\Report_Figures\GE-692006.dwg

URS

Upper Llagas Creek Flood Protection Project
Santa Clara County, CA

REACH LOCATION PLAN

NOT TO SCALE
Figure
3
5/19/06



ALL PROPERTY AND EASEMENT LINE WORK IS AT PLANNING LEVEL ACCURACY.

FIGURES 4 THROUGH 27 USE THE "ARMY CORPS OF ENGINEERS HYDRAULIC STATIONING."

FIGURE CODE	SHEET DESCRIPTION
4	SITE MAP, SHEET INDEX AND LEGENDS
5	REACH 4 – SOIL BORING SITE PLAN
6	REACH 4 – SOIL BORING SITE PLAN
7	REACH 4 – SOIL BORING SITE PLAN
8	REACH 4 AND 5 – SOIL BORING SITE PLAN
9	REACH 5 AND 6 – SOIL BORING SITE PLAN
10	REACH 6 – SOIL BORING SITE PLAN
11	REACH 6 – SOIL BORING SITE PLAN
12	REACH 6 – SOIL BORING SITE PLAN
13	REACH 6 – SOIL BORING SITE PLAN
14	REACH 6 – SOIL BORING SITE PLAN
15	REACH 6 AND 7A – SOIL BORING SITE PLAN
16	REACH 7A – SOIL BORING SITE PLAN
17	REACH 7A – SOIL BORING SITE PLAN
18	REACH 7B – SOIL BORING SITE PLAN
19	REACH 7B – SOIL BORING SITE PLAN
20	REACH 8A – SOIL BORING SITE PLAN
21	REACH 8B – SOIL BORING SITE PLAN
22	REACH 14 – SOIL BORING SITE PLAN
23	REACH 14 – SOIL BORING SITE PLAN
24	REACH 14 – SOIL BORING SITE PLAN
25	REACH 14 – SOIL BORING SITE PLAN
26	REACH 14 – SOIL BORING SITE PLAN
27	REACH 14 – SOIL BORING SITE PLAN
28	SURVEY COORDINATES – SOIL BORING LOCATIONS

SYMBOL LEGEND

DRILLED SOIL TEST SAMPLE

OW # OBSERVATION WELL

LINE TYPE LEGEND

EXISTING PROPERTY LINE

EXISTING EASEMENT LINE

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:	SCALE	PROJECT NUMBER
UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES SITE MAP, SHEET INDEX AND LEGEND	N.T.S.	26174053
	VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 4

USERNAME: josowhit Tue 18 Jul 2006 09:23am
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-04

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 08-01-02

Santa Clara Valley Water District

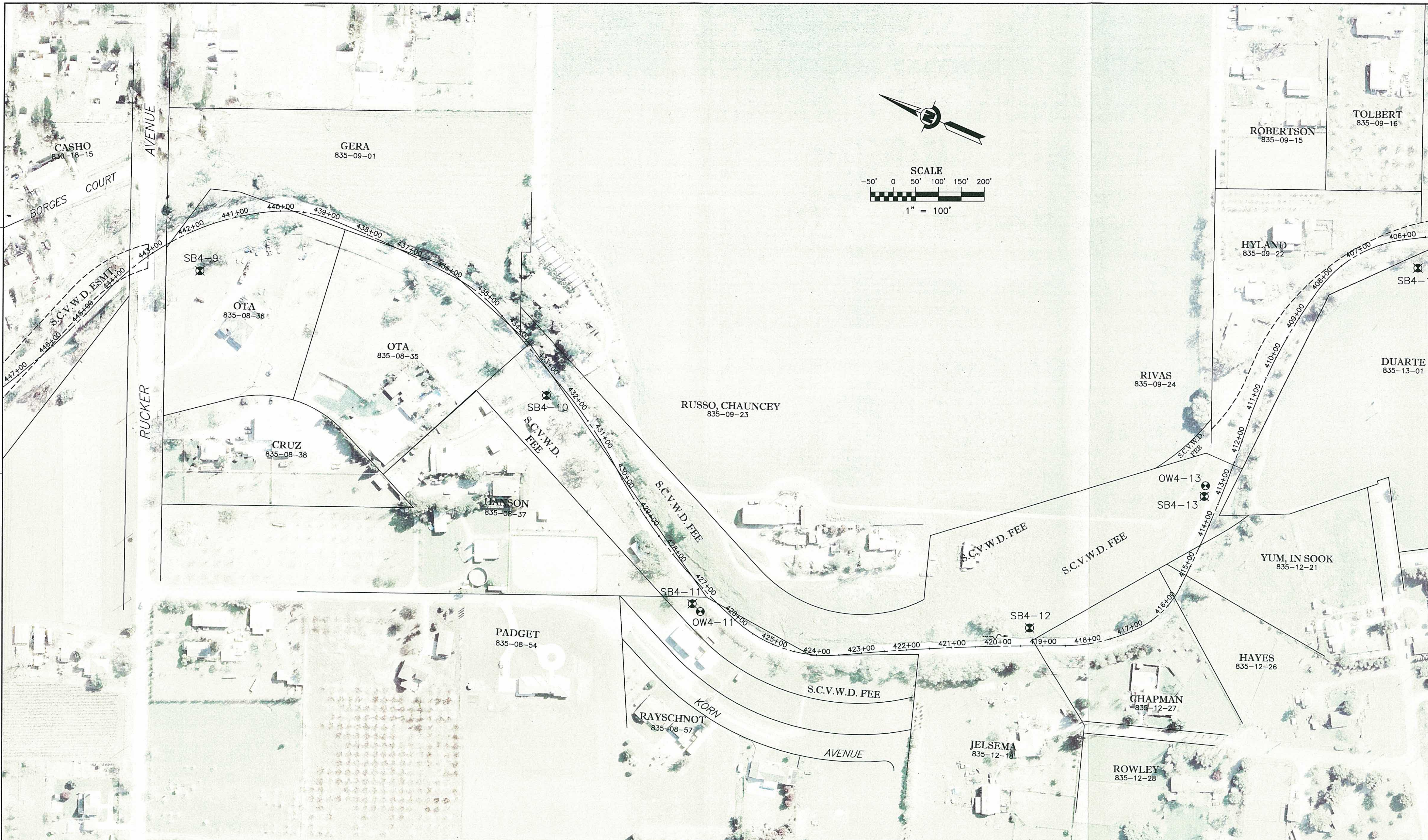
PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 4
SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 5

USERNAME: jasowhit Tue 18 Jul 2006 02:45pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-05



REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 08-01-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 4

SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 6

USERNAME: josowhit Mon 17 Jul 2006 11:34am
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-06

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 4
SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 7

USERNAME: jasowhit Mon 17 Jul 2006 11:57am
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-07

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

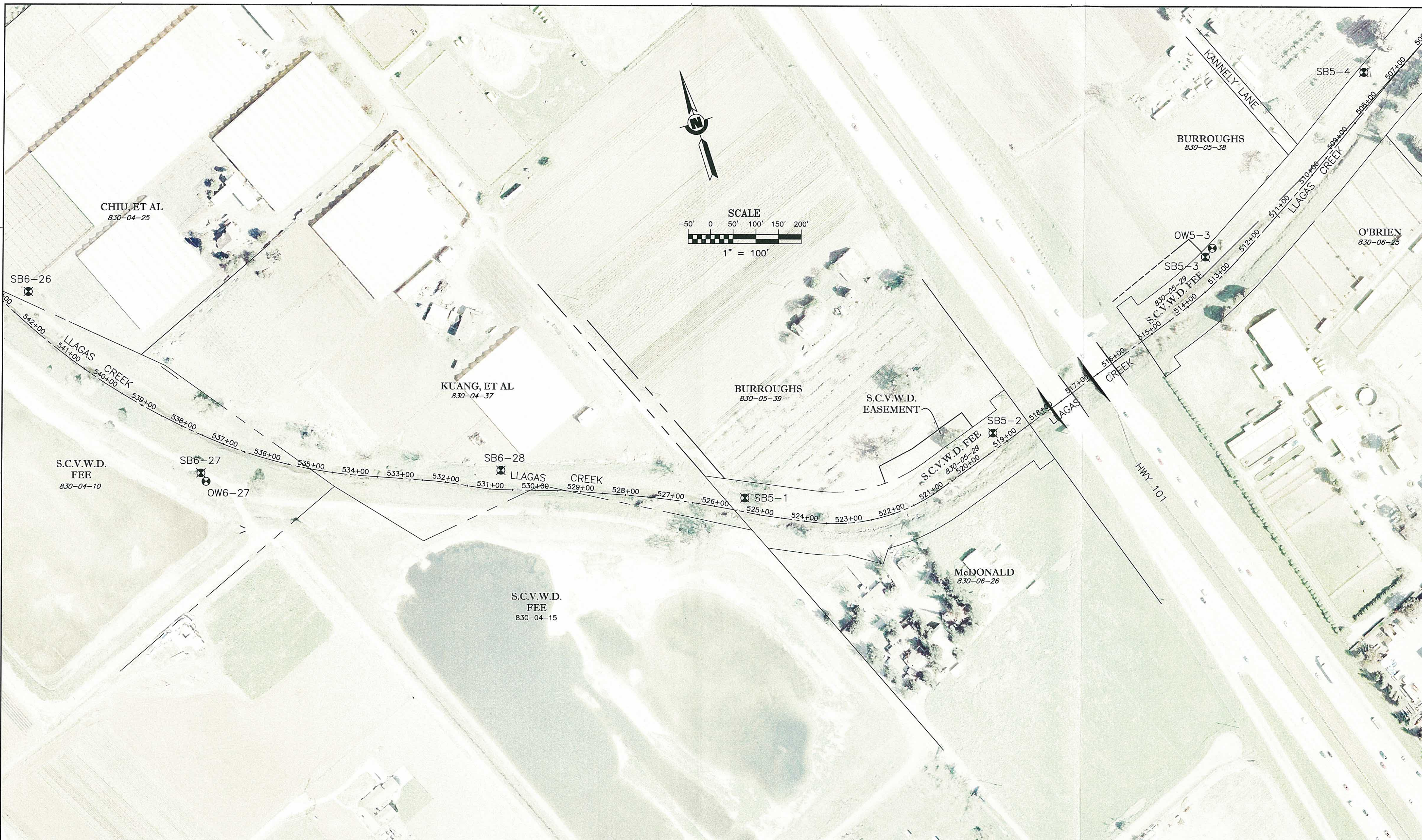
PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 4 AND 5 SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES: 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 8

USERNAME: joscwhit Mon 17 Jul 2006 01:03pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-08



REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:	SCALE	PROJECT NUMBER
UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES REACH 5 AND 6 SOIL BORING SITE PLAN	1" = 100' VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	26174053 FIGURE 9

USERNAME: jasewhit Mon 17 Jul 2006 01:07pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-09

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:	SCALE	PROJECT NUMBER
UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES REACH 6 SOIL BORING SITE PLAN	1" = 100'	26174053
	VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 10

USERNAME: josowhit Mon 17 Jul 2006 01:17pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-10

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

USERNAME: jasowhit Mon 17 Jul 2006 01:17pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-11

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

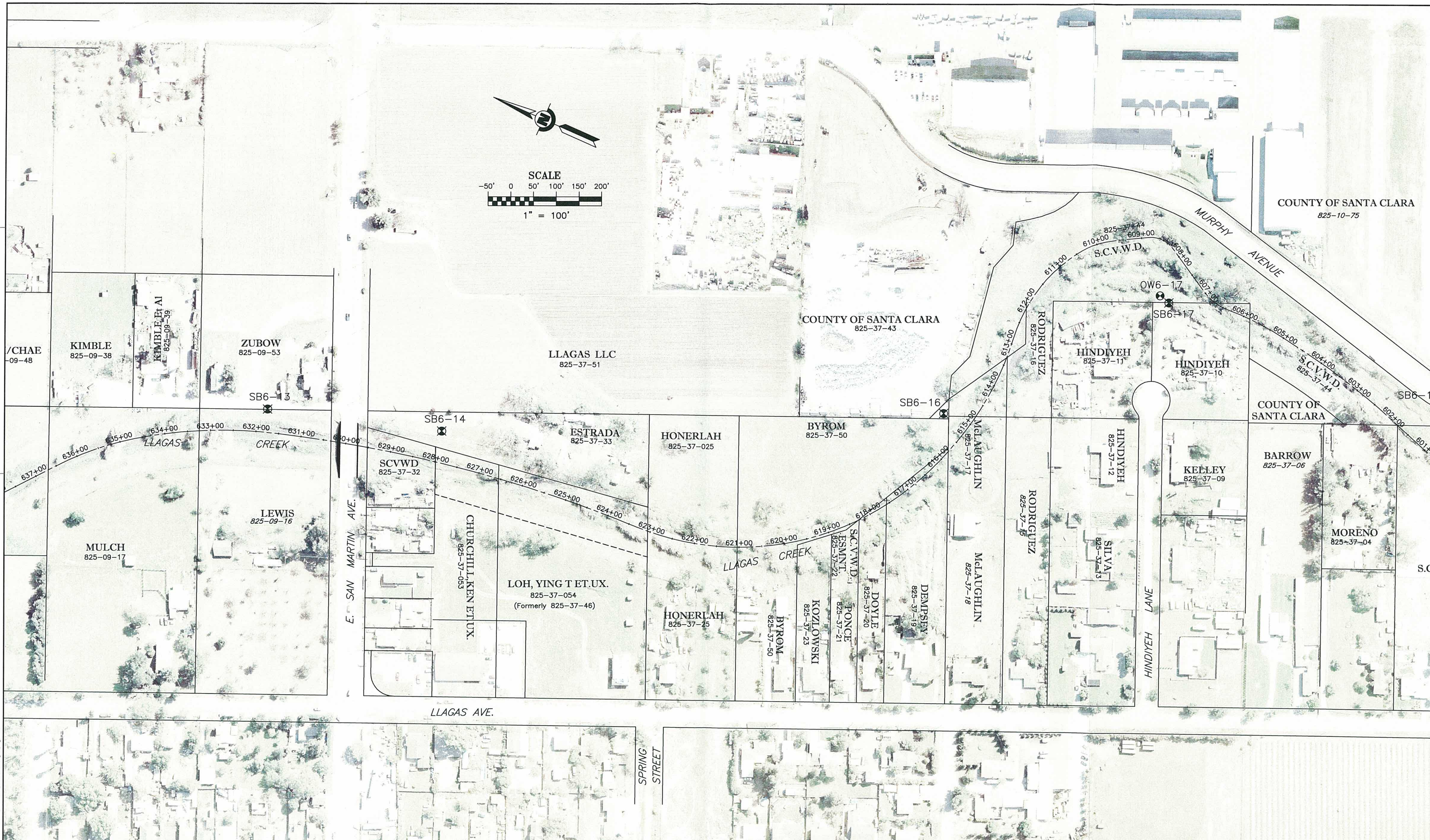
UPPER LLAGAS CREEK
TECHNICAL STUDIES
REPORT OF FIELD ACTIVITIES
REACH 6
SOIL BORING SITE PLAN

SCALE: 1" = 100'

VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING
IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

PROJECT NUMBER: 26174053

FIGURE 11



REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 6
SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES: 0 — 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 12

USERNAME: josowhit Mon 17 Jul 2006 01:26pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-12

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

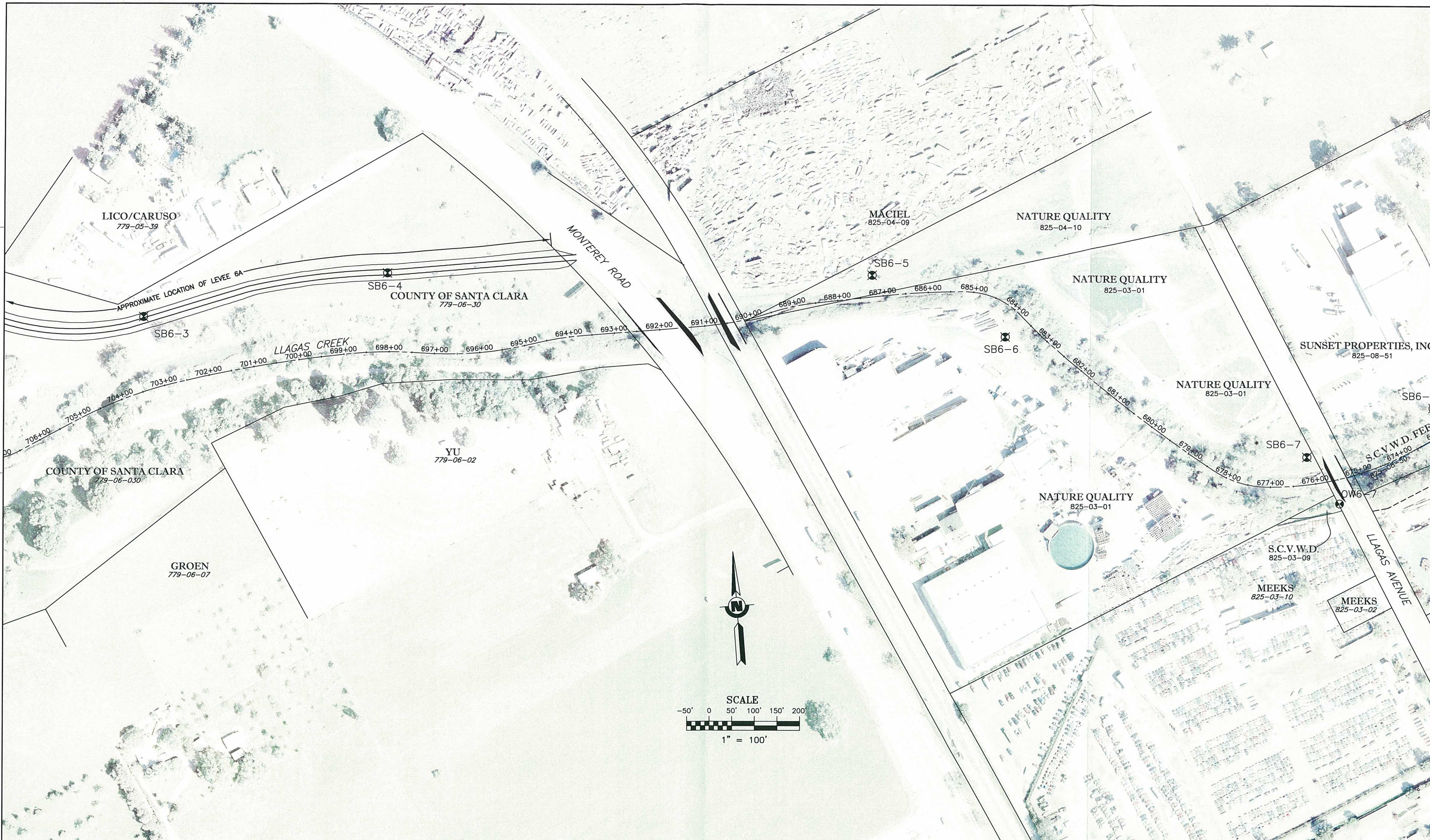
USERNAME: jasowhit Mon 17 Jul 2006 01:29pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-13

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK
TECHNICAL STUDIES
REPORT OF FIELD ACTIVITIES
REACH 6
SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 13



REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 6
SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 14

USERNAME: josowhit Mon 17 Jul 2006 01:33pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-14

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 6 AND 7A SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES 0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	FIGURE 15

USERNAME: jasowhit Mon 17 Jul 2006 01:39pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-15

REFERENCE INFORMATION AND NOTES:

1. GIS IMAGE DATE: 07-09-02

Santa Clara Valley Water District

PROJECT NAME AND SHEET DESCRIPTION:

UPPER LLAGAS CREEK TECHNICAL STUDIES REPORT OF FIELD ACTIVITIES

REACH 7A

SOIL BORING SITE PLAN

SCALE	PROJECT NUMBER
1" = 100'	26174053
VERIFY SCALES	FIGURE
0 1" BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY	16

USERNAME: josowhit Mon 17 Jul 2006 01:41pm
FILENAME: K:\Active Projects\MP-26174051\Sheets\Soil_Borings\Fig-16