FOR CONTRACT NO.: 04-1SS024

INFORMATION HANDOUT

WATER QUALITY

WATER QUALITY INFORMATION HANDOUT

MATERIALS INFORMATION

FOUNDATION REPORT
May 13, 2013

SLOPE INCLINOMETERS AND PIEZOMETERS INFORMATION

ROUTE: 04-ALA-13, PM8.3

WATER QUALITY INFORMATION HANDOUT

WATER QUALITY INFORMATION HANDOUT
CONTRACT NO. 1SS024

Project ID 04-00020868

Storm Damage Project to Repair Damages on the East Side Slope and on the Pavement Structural Section of the NB On-Ramp to ALA-13, PM 8.3 in Oakland

For Contractor

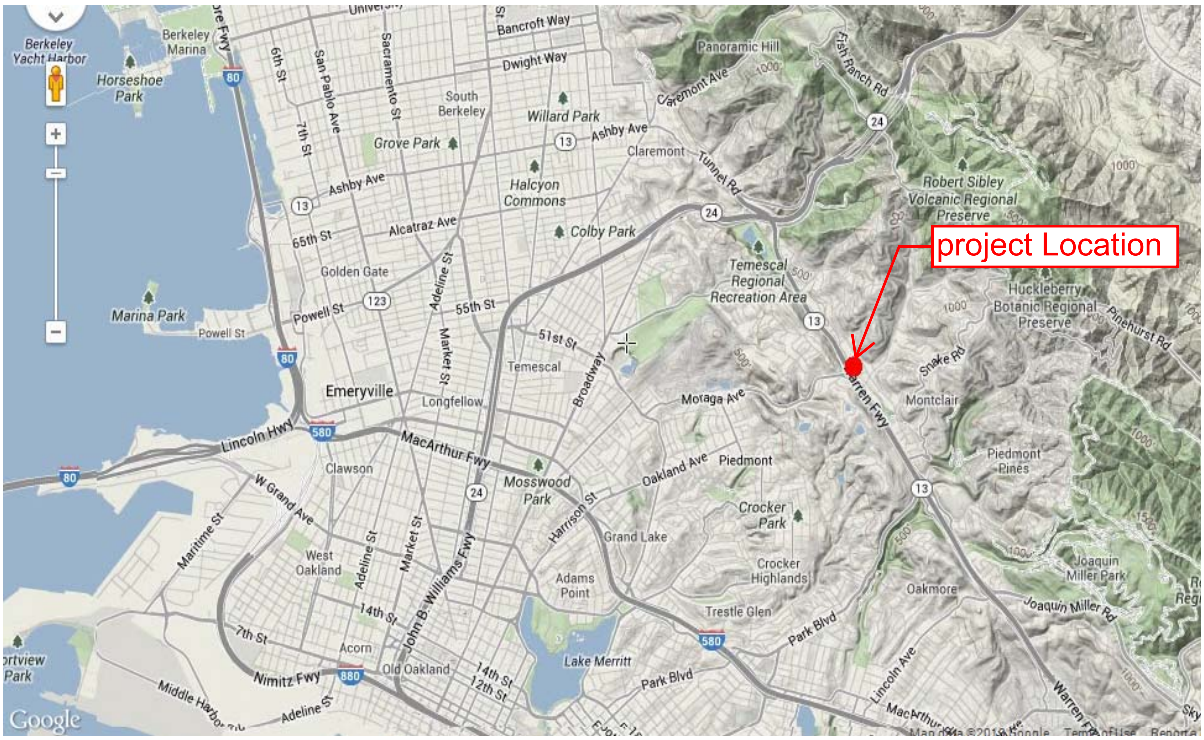
California Department of Transportation
District 4, 111 Grand Avenue
Oakland, CA 94612

Disclaimer

A “Disclaimer” is required specifying that the information provided in the Storm Water Information Handout is just a guideline and is to be used for information purposes only and should not be considered a sole source document to adhere to the requirements of the new National Pollutant Discharge Elimination System (NPDES) Construction General Permit (CGP), Number CAS000002, adopted on September 2, 2009. The contractor is required to provide water quality monitoring, sampling and implement best management practices (BMPs) based on standard industry operations, field conditions and conditions encountered based on the contractor’s means and methods. The information in this handout is not to be construed in any way as a waiver of the provisions in the CGP. Bidders and contractors are cautioned to make independent investigations and examinations as they deem necessary to satisfy the conditions encountered in performance of work, with respect to the following: sampling and monitoring locations, distribution of watershed areas for sizing of BMPs, and selection of BMPs in order to conform to the requirement of the contract documents and the CGP.

VICINITY MAP

PROJECT LOCATION MAP



RISK ASSESSMENT

	A	B	C
1	**Sediment Risk Factor Worksheet**		**Entry**
2	**A) R Factor**		
3	Analyses of data indicated that when factors other than rainfall are held constant, soil loss is directly proportional to a rainfall factor composed of total storm kinetic energy (E) times the maximum 30-min intensity (I30) (Wischmeier and Smith, 1958). The numerical value of R is the average annual sum of EI30 for storm events during a rainfall record of at least 22 years. "Isoerodent" maps were developed based on R values calculated for more than 1000 locations in the Western U.S. Refer to the link below to determine the R factor for the project site.		
4	http://cfpub.epa.gov/npdes/stormwater/LEW/lewCalculator.cfm		
5	**R Factor Value**		40
6	**B) K Factor (weighted average, by area, for all site soils)**		
7	The soil-erodibility factor K represents: (1) susceptibility of soil or surface material to erosion, (2) transportability of the sediment, and (3) the amount and rate of runoff given a particular rainfall input, as measured under a standard condition. Fine-textured soils that are high in clay have low K values (about 0.05 to 0.15) because the particles are resistant to detachment. Coarse-textured soils, such as sandy soils, also have low K values (about 0.05 to 0.2) because of high infiltration resulting in low runoff even though these particles are easily detached. Medium-textured soils, such as a silt loam, have moderate K values (about 0.25 to 0.45) because they are moderately susceptible to particle detachment and they produce runoff at moderate rates. Soils having a high silt content are especially susceptible to erosion and have high K values, which can exceed 0.45 and can be as large as 0.65. Silt-size particles are easily detached and tend to crust, producing high rates and large volumes of runoff. Use Site-specific data must be submitted.		
8	Site-specific K factor guidance		
9	**K Factor Value**		0.37
10	**C) LS Factor (weighted average, by area, for all slopes)**		
11	The effect of topography on erosion is accounted for by the LS factor, which combines the effects of a hillslope-length factor, L, and a hillslope-gradient factor, S. Generally speaking, as hillslope length and/or hillslope gradient increase, soil loss increases. As hillslope length increases, total soil loss and soil loss per unit area increase due to the progressive accumulation of runoff in the downslope direction. As the hillslope gradient increases, the velocity and erosivity of runoff increases. Use the LS table located in separate tab of this spreadsheet to determine LS factors. Estimate the weighted LS for the site prior to construction.		
12	LS Table		
13	**LS Factor Value**		8.105
14			
15	**Watershed Erosion Estimate (=RxKxLS) in tons/acre**	119.954	
16	**Site Sediment Risk Factor**	**High**	
17	Low Sediment Risk: < 15 tons/acre		
18	Medium Sediment Risk: >=15 and <75 tons/acre		
19	High Sediment Risk: >= 75 tons/acre		
20			

Receiving Water (RW) Risk Factor Worksheet	Entry
A. Watershed Characteristics	**yes/no**
A.1. Does the disturbed area discharge (either directly or indirectly) to a **303(d)-listed waterbody impaired by sediment** (For help with impaired waterbodies please visit the link below) or has a **USEPA approved TMDL implementation plan for sediment**?: http://www.waterboards.ca.gov/water_issues/programs/tmdl/integrated2010.shtml **OR** A.2. Does the disturbed area discharge to a waterbody with designated beneficial uses of SPAWN & COLD & MIGRATORY? (For help please review the appropriate Regional Board Basin Plan) http://www.waterboards.ca.gov/waterboards_map.shtml	**no**

Region 1 Basin Plan

Region 2 Basin Plan

Region 3 Basin Plan

Region 4 Basin Plan

Region 5 Basin Plan

Region 6 Basin Plan

Region 7 Basin Plan

Region 8 Basin Plan

Region 9 Basin Plan

Combined Risk Level Matrix

<table>
<tr><th></th><th></th><th colspan="3">Sediment Risk</th></tr>
<tr><th></th><th></th><th>Low</th><th>Medium</th><th>High</th></tr>
<tr><td rowspan="2">Receiving Water Risk</td><td>Low</td><td>Level 1</td><td colspan="2">Level 2</td></tr>
<tr><td>High</td><td colspan="2">Level 2</td><td>Level 3</td></tr>
</table>

Project Sediment Risk:	**High**
Project RW Risk:	**Low**
Project Combined Risk:	**Level 2**

THE CALCULATION OF "R" FACTOR

Figure 1-Erosivity Index Zone Map

The EI distribution zone is 24

Table 1-Erosivity Index Table

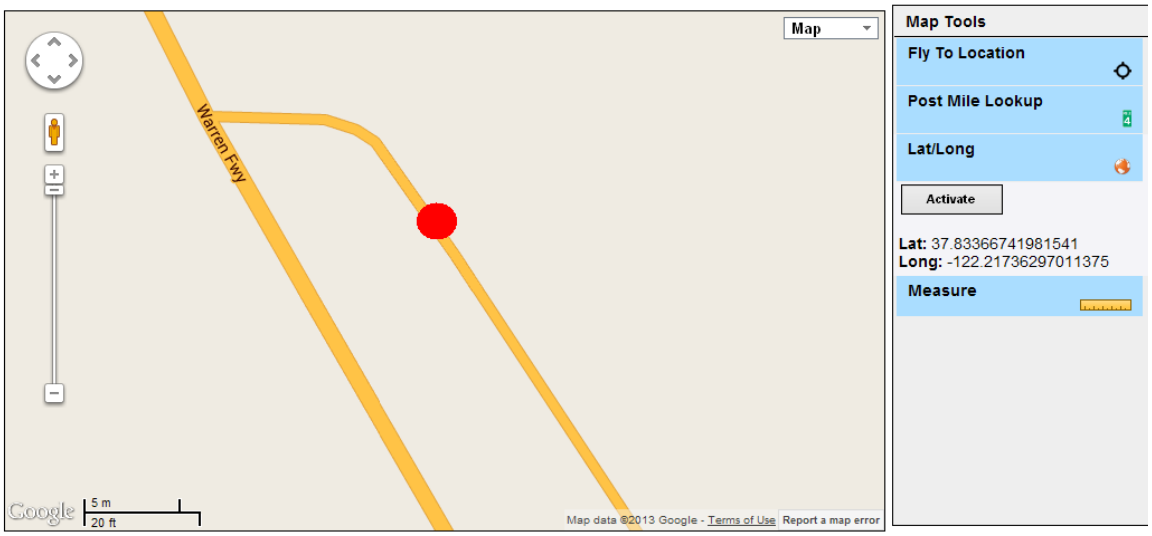
EI percentage (Aug 26, 2014 to Aug 25, 2015)

100%

Figure 4_ Isoerodent map of California =40

R factor is 100 X 40% = 40

K and LS



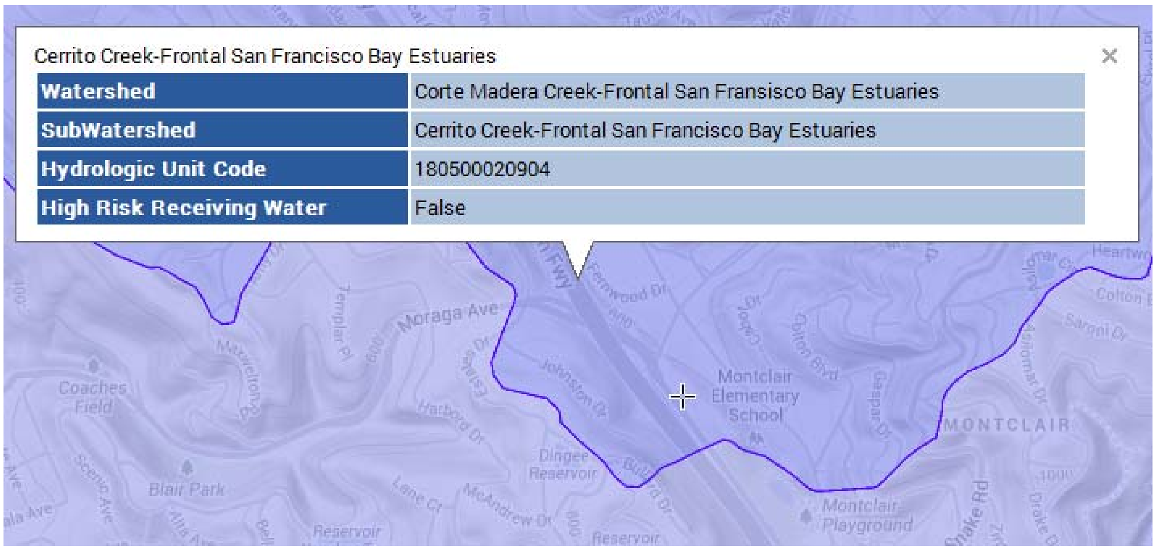
Populate K Factor is 0.37

Populate LS Factor is 8.105

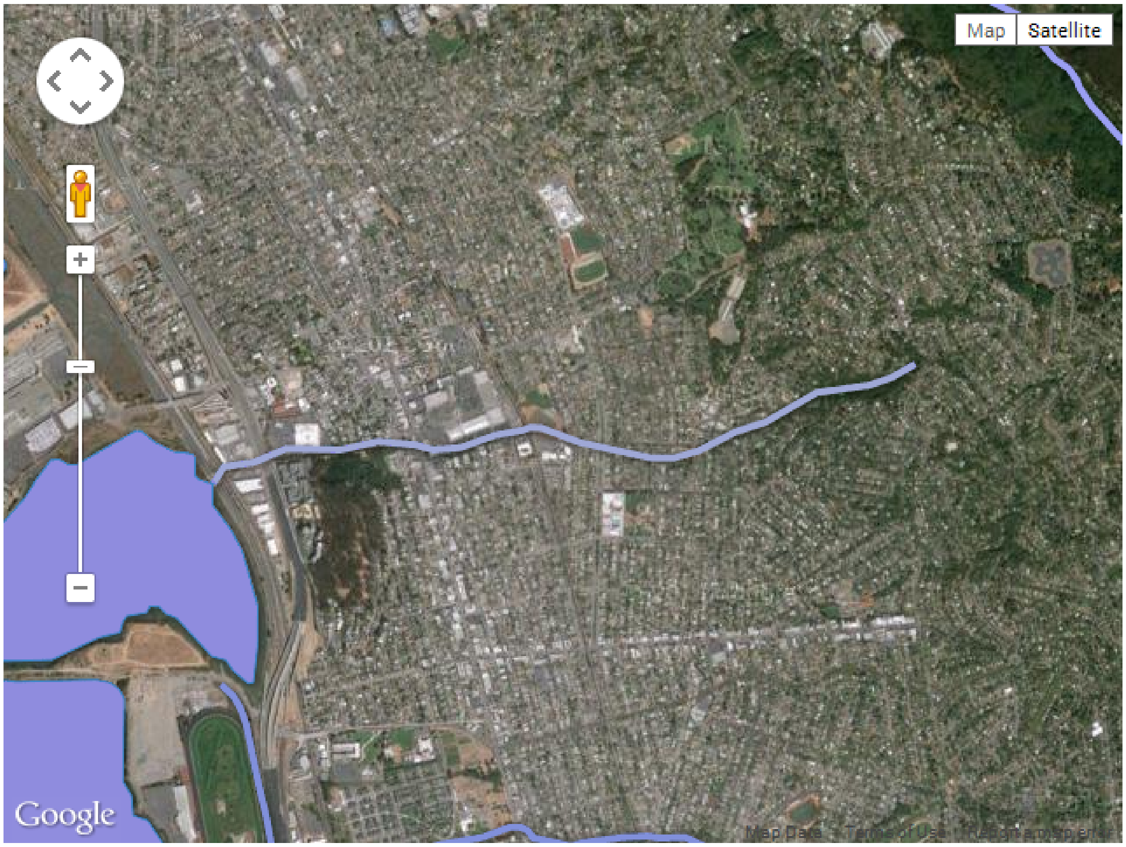
RECEIVING WATER RISK: LOW

The project area discharges to the Cerrito Creek and eventually discharges to San Francisco Bay Central. Both Cerrito Creek and San Francisco Bay Central are not a 303(d)-listed waterbody impaired by sediment or has a USEPA approved TMDL implementation plan for sediment.

Both are not a water body with designated beneficial uses of SPAWN & COLD & MIGRATORY



Cerrito Creek



TMDLS & 303(D) LIST (2010) FOR CERRITO CREEK

Key: Pollutant on 303(d) list | Pollutant with a TMDL

Pollutant	Source	Size	Status	Comments
Trash	Illegal dumping	2 Miles	TMDL required	
Trash	Urban Runoff/Storm Sewers	2 Miles	TMDL required	

CENTRAL BASIN

COUNTY Waterbody	AGR	MUN	FRSH	GWR	IND	PROC	COMM	SHELL	COLD	EST	MAR	MIGR	RARE	SPWN	WARM	WILD	REC-1	REC-2	NAV
ALAMEDA COUNTY, continued																			
Temescal Creek									E						E	E	E	E	
Claremont Creek															E	E	E	E	
Strawberry Creek															E	E	E	E	
Codornices Creek									E			E	E	E	E	E	E	E	
Village Creek															E	E	E	E	
Capistrano Creek															E	E	E	E	
CONTRA COSTA COUNTY																			
Cerrito Creek															E	E	E	E	
Baxter Creek															E	E	E	E	
Richmond Inner Harbor							E			E						E	E	E	E

CENTRAL BASIN

COUNTY Waterbody	AGR	MUN	FRSH	GWR	IND	PROC	COMM	SHELL	COLD	EST	MAR	MIGR	RARE	SPWN	WARM	WILD	REC-1	REC-2	NAV
SAN FRANCISCO COUNTY																			
Golden Gate Channel							E				E	E	E	E		E	E	E	E
San Francisco Bay Central					E	E	E	E		E		E	E	E		E	E	E	E
Crissy Field Lagoon										E						E	E	E	
Golden Gate Park Lakes															E	E		E	
Lobos Creek		E												E	E	E	E	E	
Mountain Lake															E	E	E	E	
MARIN COUNTY																			
San Rafael Creek									E						E	E	E	E	E
Corte Madera Creek							E		E			E	E	E	E	E	E	E	E
Larkspur Creek									E				E	E	E	E	E	E	
Tamalpais Creek									E			E	E	E	E	E	E	E	
Ross Creek (Marin)									E			E	E	E	E	E	E	E	
Phoenix Lake		E					E		E				E	E	E	E	E*	E	
Phoenix Creek			E						E						E	E	E	E	
Bill Williams Creek			E						E					E	E	E	E	E	

RAINFALL DATA

Rainfall Intensity can be obtained by the following link:

http://www.wrcc.dri.edu/pcpnfreq/nca5y24.gif

Refer to Chapters 800, Highway Drainage Design of Highway Design Manual for information on runoff coefficient and shed map.

CONCEPTUAL SAMPLING LOCATION PLAN

Note: The sampling location is conceptual and for reference only. The exact sampling should be determined by the Contractor based on the field condition and work phases.

Discharge Point and Recommended Sampling Location

Recommended Control Sampling Location

NOTE:
FOR ACCURATE RIGHT OF WAY DATA, CONTACT RIGHT OF WAY ENGINEERING AT THE DISTRICT OFFICE.

LEGEND:

- No. — STRUCTURAL SECTION No.
- (No.) — CURVE DATA No.
- MIDWEST GUARDRAIL SYSTEM (MGS)
- TIEBACK RETAINING WALL
- CHAIN LINK FENCE

CURVE DATA

No.	R	Δ	T	L
1	4000'	02° 15' 27"	78.81'	157.60'
2	3000'	01° 38' 53"	43.15'	86.29'
3	300'	-17° 37' 20"	46.50'	92.27'

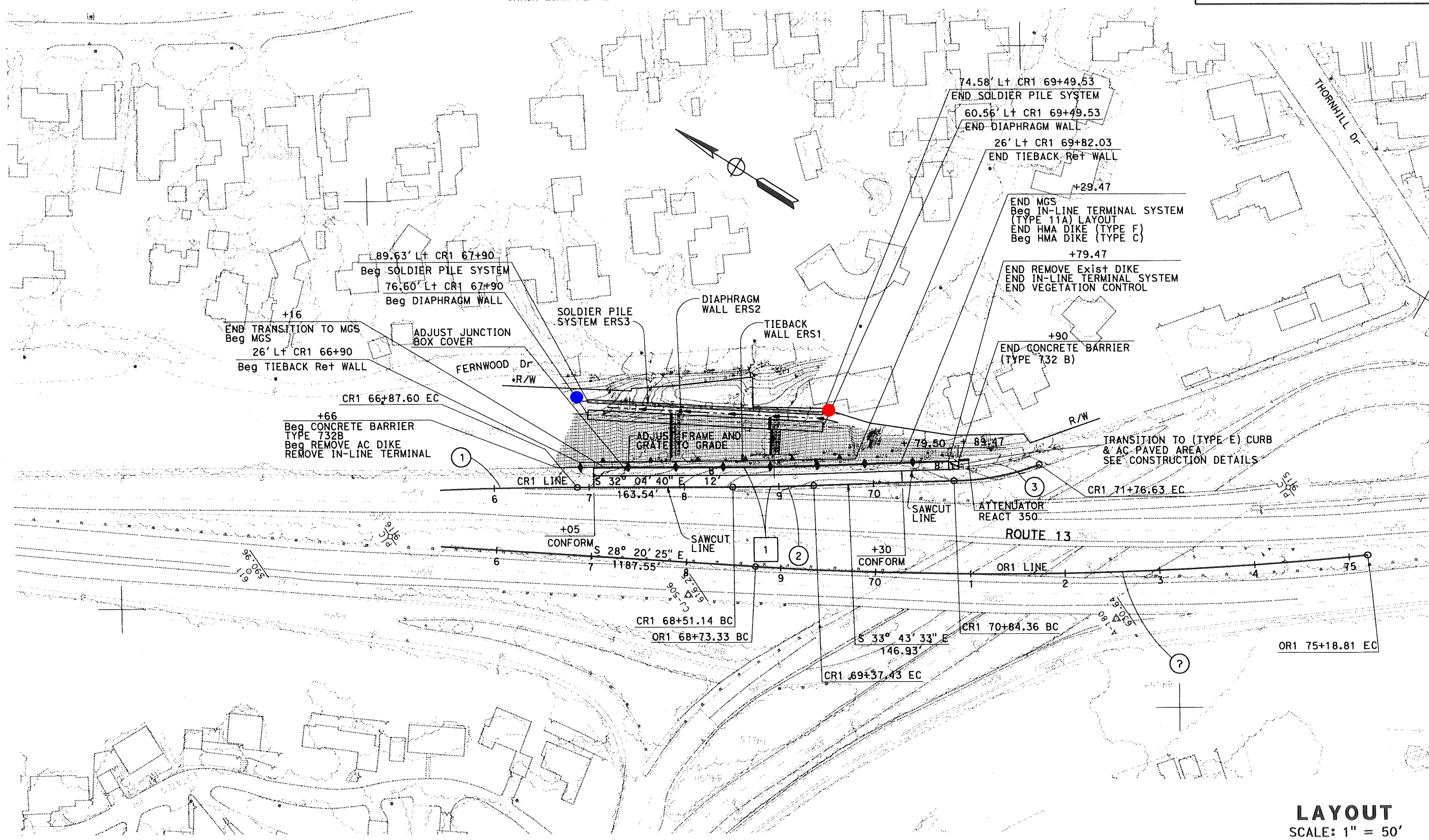
Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Ala	13	8.3		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.



LAYOUT
SCALE: 1" = 50'

L-1

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans
DESIGN

FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	CHECKED BY	REVISED BY	DATE REVISED

LAST REVISION 11-15-13 DATE PLOTTED => 18-NOV-2013 TIME PLOTTED => 14:13

BORDER LAST REVISED 7/2/2010
USERNAME => s126849
DGN FILE => 0400020868ea001.dgn
RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3
UNIT 0735
PROJECT NUMBER & PHASE 04000208681

FOUNDATION REPORT

Memorandum

Flex your power!
Be energy efficient!

To: MR. MUTHANNA OMRAN
Bridge Design Branch Chief
Office of Structures Design – West

Attention: Paotsan Wang

Date: May 13, 2013

File: 04-ALA-13 PM 8.3
04-1SS020
Efis # 0400020868-1
Thornhill/Fernwood Landslide

From: EDUARDO ORTEGA
Associate Materials and Research Engineer
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

MAHMOOD MOMENZADEH
Chief, Branch C
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

RIFAAT NASHED
Engineering Geologist
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

Subject: Foundation Report

I. INTRODUCTION

This Foundation Report provides our geotechnical recommendations to repair a landslide located on the Thornhill Dr on-ramp to NB Route 13 in Montclair District, City of Oakland, Alameda County, California.

II. BACKGROUND

The initial slide occurred on February 7, 1973 and extended from the east shoulder of NB Route 13 on-ramp from Thornhill Drive to the West shoulder of Fernwood Drive (see attached Location and Vicinity Maps). An investigation was performed at that time for which 8 exploratory borings were drilled (2 with slope inclinometers). The constructed emergency repair system consisted of a combination of pumped wells in the median behind the slide, a 5 foot deep trench underdrain adjacent to Fernwood Drive and an earth buttress at the toe of the slide. The earth buttress extended 20 feet beyond our Right-of-Way and encroached into the southbound shoulder and lane of Fernwood Drive.

The emergency measures performed well, halting further slide movement until 1983, when two consecutive years of abnormally heavy rainfall reactivated the slide, causing cracking and settlement along the on-ramp shoulder and heaving in Fernwood Drive. Drainage wells installed in the median were pumped on a regular basis for about 10 weeks. Slide movement ceased soon after the pumping began.

On March 12, 1986, renewed slide movement of a magnitude almost as great as the original failure occurred, causing up to 1 foot of uplift in Fernwood Drive, and up to 3 feet of vertical movement at the top of the slide. Similar to 1983 mitigation, the median wells were pumped and slide movement ceased. A supplemental investigation was performed including an additional boring (with slope inclinometer) and a stability analysis using the computer program STABL. As a result of these new studies, a permanent slide repair plan was developed which consisted of:

- Unload the slide by excavating the upper portion of the embankment and replace it with lightweight fill.
- Drain the slide with a combination of a deep underdrain trench, a drainage blanket, a drainage gallery (horizontal drains) and an underdrain near the existing house in Fernwood Drive.

Due to renewed slide movements, during November and December 2004, we drilled four boreholes 60 feet deep each and installed slope inclinometer in all boreholes along Thornhill Drive on-ramp to Route 13. In June 2005, we drilled 2 additional boreholes 60 feet deep each and installed 2 slope inclinometers on the buttress behind the existing retaining wall on Fernwood Drive. Since then, we have been monitoring the slope movement. See Sect IV. Landslide Monitoring for slide condition based on SI readings.

On January 27, 2012 we received a request from Central Region Project Development to prepare a Foundation Report with recommendations to repair the slide at this site.

III. GEOLOGY

III.1 Topography

The project site is located in the Montclair District, within Berkeley Hills west of Temescal Creek, northeast of the City of Piedmont. The slide is located within the rift valley of the Hayward Fault (See attached Fault Map). The project elevation ranges between 558 ft to 602 ft above sea level.

III.2 Site Geology

The project area is located in the Franciscan Complex which represents one of the two intricately deformed Mesozoic rock complexes of Alameda County. This complex represents the accreted and deformed remnants of Jurassic oceanic crust and overlying volcanic arc plus a thick sequence of turbidities.

The project site is entirely covered by undivided surficial deposits (Qu) of Holocene and Pleistocene age, overlaying Franciscan mélange (KJfm) (See attached Geology Map). The Franciscan mélange of late Jurassic and/or late Cretaceous, consists of sheared black argillite, greywacke sandstone, and minor green tuff, containing blocks and lenses of meta-graywacke, chert, shale, metachert, serpentinite, greenstone, amphibolites, tuff, eclogite, quartz schist, greenschist, basalt, marble, conglomerate, and glaucophane schist. Blocks range in size from pebbles to several hundred meters in length (USGS, OFR 96-252)[1]

The project site is located within the Alquist - Priolo map zoning of the Hayward Fault (See attached Fault Map). Also, the project site is located 9.1 miles (14.7 km) west of the Calaveras Fault, and 18.8 miles (30.3 km) east of the San Andreas Fault. The Hayward Fault, the San Andrea Fault, and the Calaveras Fault are active faults with Maximum Magnitude (MMax) of 7.3, 7.9, and 7.4, respectively. According to Association of Bay Area Government (ABAG), the project area is exposed to a very violent level of ground shaking, and classified as having a moderate level of liquefaction. The site may also be subjected to surface rupture.

IV. FIELD EXPLORATION AND SUBSURFACE CONDITION

IV.1 Field Exploration

Up to date, a total of 17 boreholes were drilled in the project area ranging between 53 ft and 86 ft depth below the existing ground surface at these boreholes as summarized below:

- 8 exploratory borings (P-7, P-8, P-9, P-10, P-11 & P-12 on median of Hwy 13) and (P-13 & P14 west of Fernwood Drive)) with 4 slope inclinometers (SI-3, SI-4, SI-5 &SI-6) were drilled between 1973 and 1986.
- 4 boreholes P-1, P-2, P-3 & P-4 (renamed R-04-001, R-04-002, R-04-003 and R-04-004 respectively) were drilled along Thornhill Drive on-ramp to Route 13 to the depth of 61.0 ft below the ground surface in December 2004. And 4 slope inclinometers (SI-7, SI-8, SI-9B & SI-10) were installed in these boreholes.

[1]R.W. Graymer, D.L. Jones, and E.E.Brabb, 1996, Preliminary Geologic Map Emphasizing Bedrock Formation in Alameda County , California, USGS-OFR 96-252.

- 2 boreholes P-5 and P-6 were drilled on the buttress behind the existing retaining wall on Fernwood Drive in January 2005 to the depth of 60 feet below the existing ground surface, and 2 slope inclinometers SI-11 and SI-12 were installed in these boreholes, boreholes were not logged so they are not shown on the LOTB.
- 3 boreholes R-12-001, RH-12-002 and R-12,003 were drilled on June 2012 to the depth of 60 ft.

The locations of these boreholes are shown on the LOTBs attached to this report.

IV.2 Subsurface Conditions

The soil material encountered at these boreholes consist of 15 feet soft to medium stiff grayish brown sandy/silty lean clay with shale fragments, underlain by 33 to 35 feet stiff to very stiff greenish sandy lean clay. Decomposed shale to shale (mélange) underlies this succession at depth ranging between 48 to 50 feet below the Hwy 13 ground surface.

It should be noted that a portion of the road embankment at the top of the slide was previously excavated and replaced by lightweight (sawdust) fill to mitigate the slide.

IV.3 Groundwater

The groundwater elevation has been continuously monitored in five piezometers installed at borings P-8, P-9, P-10, P-11 and P-12 which drilled in the median of the SR 13 at the top of the landslide. The monitoring was conducted from January 2005 to November 2011. The groundwater elevation ranges between 601.6 feet elevation (0.4 ft depth, below the ground surface) at borehole P-9 in March 31, 2006, and 569.55 feet elevation (32.45 ft depth below the ground surface) at borehole P-11 in September 15, 2008.

V. LANDSLIDE MONITORING

Readings from the slope inclinometers installed in 2004 indicate ground movements extend down to 9 feet depth at the location of the SI and 25 ft at the location of Wall 1 below the roadway grade of Route 13 at Thornhill Drive on-ramp. Readings from the inclinometers installed in 2005 indicate ground movements extending down to 30 feet depth below roadway grade of Fernwood Drive.

VI. SLOPE STABILITY ANALYSES

Back-analysis

Soil/rock strength parameters of the sliding mass were determined using back-analysis of the landslide. This is necessary because it is very difficult to estimate the residual shear strength of the soil/rock at the slip-plane by conventional methods. The size of the sliding mass was estimated using slip-plane location (from SI data), head scarp location, and other geologic features. In dry condition the slope has a Safety Factor (SF) of 1.11. Since the landslide is creeping at very slow rate, we believe that the landslide currently has a Safety Factor (SF) of slightly less than 1.0. Our back analyses show that slide soil material has an effective friction angle of 10° and cohesion of 350 psf (along the slip-plane) for an assumed SF of slightly less than 1.0. To simulate the faster landslide movement during the wet periods, we add pore water pressure within the sliding mass to reduce the SF to 0.95. The soil strength and pore water pressure parameters determined by back-analysis are summarized in Table 1.

The back analysis was performed using computer program "SLOPE/W". The graphical outputs (generated by the computer program) are attached.

Table 1. Back Calculated Soil/Rock Strength and Pore Water Pressure

Soil/Rock Type	Unit Weight Psf	Internal Friction Angle ϕ Degrees	Cohesion C psf	Pore Water Pressure Parameter r_u	Safety Factor (SF)
Ret. Soil	135	10	350	0	~ 1.0
Ret. Soil	135	10	350	0.22	~ 0.95

VII. FOUNDATION RECOMMENDATIONS

VII.1 Wall Types and Limits

Based on the site topography and geologic conditions, it is our opinion that the most suitable repair system combination at these locations are:

- Wall located at Thornhill Drive on-ramp to Hwy 13 (Wall 1):

  A soldier pile with tie back retaining wall approximately 10 feet in exposed height, 25 ft in slide depth, and 292 feet long to be built along Route 13 at Thornhill Drive on-ramp. The wall Starts 25' Left of Line CR1 Sta 66+90 and End 25' Left of Line OR1 Sta 69+82.

- Structures on earth buttress located at Fernwood Drive (Walls 2 and 3):

  Wall 2: A slope stressing beam wall type with two ground anchors approximately 250 ft in length to be constructed on the buttress down slope. The beam starts 75.6 ft, left of Line CR1 Sta 67+00 and End 60.56 ft, left of Line CR1 Sta 69+49.53.

  Wall 3: A soldier pile wall with a beam cap at the existing earth buttress within the bench area below the buttress down slope approximately 250 ft in length. The soldier piles will be on a 30 inches hole @ 7 ft. Soldier piles Start 90.63 ft, left of Line CR1 Sta 67+00 and End 75.58 ft, left Line CR1 Sta 69+49.96.

- The existing retaining wall at Fernwood Drive will remain undisturbed.

  Below are our recommended design parameters for each of the above walls

VII.2 Walls Geotechnical Design Parameters

Soil/Rock Parameters, Tieback loads and Pile Embedments

We performed additional slope stability analyses for this proposed repair system using a specified slip-plane and back calculated soil/rock parameters, and pore water pressure condition. The analyses performed to determine the tie back loads and the resulting safety factor (SF). Below are summary of soil/rock strength design tieback loads and soldier piles parameters.

Table 2. Rock/Soil strength parameter

Geologic Unit	Unit Weight (γ) pcf	Friction angle (°) Degree	Cohesion (c) psf
Retained Soil	135	10	350
Foundation Soil	135	35	0

Table 3. TIEBACK LOADS

TYPE OF LOADING	SF	Tieback load On Retaining Wall 1	Tieback load, Row 1 Wall 2	Tieback load , Row 2 Wall 2
Static	1.337	15.8 Kips/ft @ 15° angle	12 Kips/ft @ 40° angle	12 Kips/ft @ 60° angle
Seismic	> 1.0	23.75 Kips/ft @ 15° angle	18 Kips/ft @ 40° angle °	18 Kips/ft @ 60° angle

The results of the stability analysis are attached

Based on the results of the above analyses, we recommend a static design tieback load of 15 kips/ft of wall for Wall1 and 12 kips/ft for each of the two tiebacks in Wall 2. Proof and performance of all tieback are required in accordance with Caltrans specifications.

Table 4: Soldier Piles, Walls 1 and 3

Wall #	Wall Height ft	Pile spacing ft	Pile diameter ft	Pile minimum embedment ft
Wall 1	10(exposed), 25 (slide)	6	3	15
Wall 3	0 (exposed) 30 (slide)	7	3	15

TABLE 5 Pile Friction and Tip Compression Capacities for Walls 1 and 3

	Ultimate Kips/sqft	Allowable Kips/sqft
Unit pile shaft friction per unit surface area of the pile length below the dredge line of the wall	3.50	1.45 (SF=2)
Pile tip compression bearing pressure per unit tip area of the CIDH pile	107.70	35.90 (SF=3)

Static and Seismic Earth Pressures

The proposed soldier piles tie back retaining wall system can be designed using the earth pressures and criteria outline in Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.7 Figure 5.5.5.7.1-1a.

For Wall 1, use an additional rectangular pressure diagram from top of the wall to a depth equal to the wall height. This is equivalent to 2 ft of fill.

For seismic earth pressure increments applied to Wall 1 and 2, use a uniform earth pressure equal to 17 H (psf) applied 0.5H above the base. For Wall 3, use a triangular seismic earth pressure distribution with the pressure at the wall base equal to 34H (psf). H is the height of the wall.

Friction Factor between wall and backfill = 2/3 of Internal Friction Angle
$(\delta = 2/3\ \varphi)$

The above recommended earth pressures are based on the assumption that an adequate drainage system will be provided to prevent the development of hydrostatic pressure behind the wall. If complete drainage of the wall cannot be achieved, add hydrostatic pressure assuming groundwater at 5 ft below top of wall.

We recommend that the entire surface of the cut slope including the bearing area of the slope stressing concrete whalers be reinforced with soil nail shoring and shotcrete surface to enable the design tieback loads be developed. The purpose of this soil nail wall reinforcement is to stabilize the cut slope during construction and to prevent excessive movement and bearing failure of the foundation soils and weathered bearing rock due to tie back loads imposed by slope stressing. In addition, we recommend that a continuous waler instead of individual waler be used at each of the proposed slope stressing levels in order to distribute the distribution of load and deformation more uniformly.

Below are the requirements for soil nail shoring:

- Use design Grade 75 ksi bars
- Ultimate punching capacity of 40 Kips.
- Soil nail Design Load of 3.25 Kips.
- Design length (embedment depth) of 25 ft min for all nails irrespective of their location.
- Soil Nail spacing should be limited to not more than 6 ft horizontal and 5 ft vertical.
- Extend the Soil Nail area 5 ft beyond the upper beam
- Soil Nails drill hole 6 inches in diameter..

In order to prevent instability of the slide and achieve the design tieback load the following order of installation shall be followed:

Construct the soil nail wall by excavating and installing the Soil Nails in top-down fashion. Then, install tiebacks and whalers. To finalize the wall, the stressing of tie backs will be bottom to top, and in each waler the stressing will follow from outside to center.

All excavation, drilling and soil nail installation shall be in conformance with the Caltrans specification for the soil nail wall construction.

VIII. CORROSION

The result of our corrosion testing is shown on table 6 below.

TABLE 6 Corrosion Results

Sample from Borehole	Depth Ft	Resistivity Ohm-cm	pH
P-2	10-15	2350	7.9
P-3	5-10	2600	7.6
P-3	10-15	2150	7.1

According to the Caltrans Corrosion Guidelines dated September 2003 (Version 1.0), for structural elements, the Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

Chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less.

For structural elements, the minimum resistivity of soil and/or water indicates the relative quantity of soluble salts present in the soil or water. In general, a minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Soil and water that have a minimum resistivity less than 1,000 ohm-cm are tested by the Chemical Testing Branch of METS for chlorides and sulfates. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm because a ***minimum resistivity greater than 1,000 ohm-cm indicates that the chloride and sulfate contents are low (i.e., low corrosion potential).***

Our site is considered non-corrosive because the samples show a resistivity greater than 1,000 ohm-cm and a pH greater than 5.5.

IX. FILL PLACEMENT AND DEBRIS REMOVAL

To construct the proposed wall 1, backfill has to be placed behind the wall to bring the fill behind the wall to the desired grade. Some excavation will also be required in front of the walls to create the required 5 ft (minimum) wide bench. We recommend that the backfill be either structural backfill or aggregate base (Class III).

X. CONSTRUCTION CONSIDERATION

Drilling difficulties for soldier beam piles, tiebacks, and soil nails are anticipated due to caving potential of drill hole in soft soil/rock zones, water and hard rock existing at the site. Drilling method and equipment shall be capable of accomplishing such variations of soil and rock characteristics.

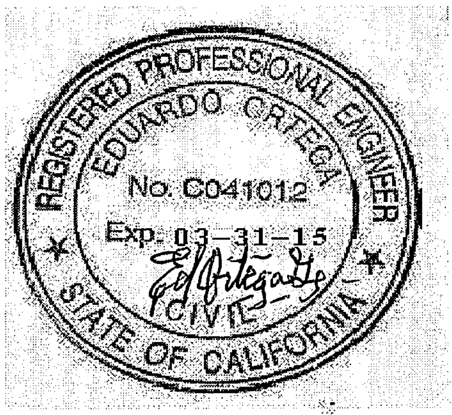
Stockpiling of the excavation material and concurrent use of heavy construction equipment on the lower bench shall be avoided especially before wall 3 is installed.

All works for Wall 2 construction shall be subsequent to completion of Wall 1 and 3.

Groundwater dewatering is anticipated for the above wall construction and shall be the responsibility of the contractor.

* * * * *

Should you have any questions, please call me at (510) 286-4821.

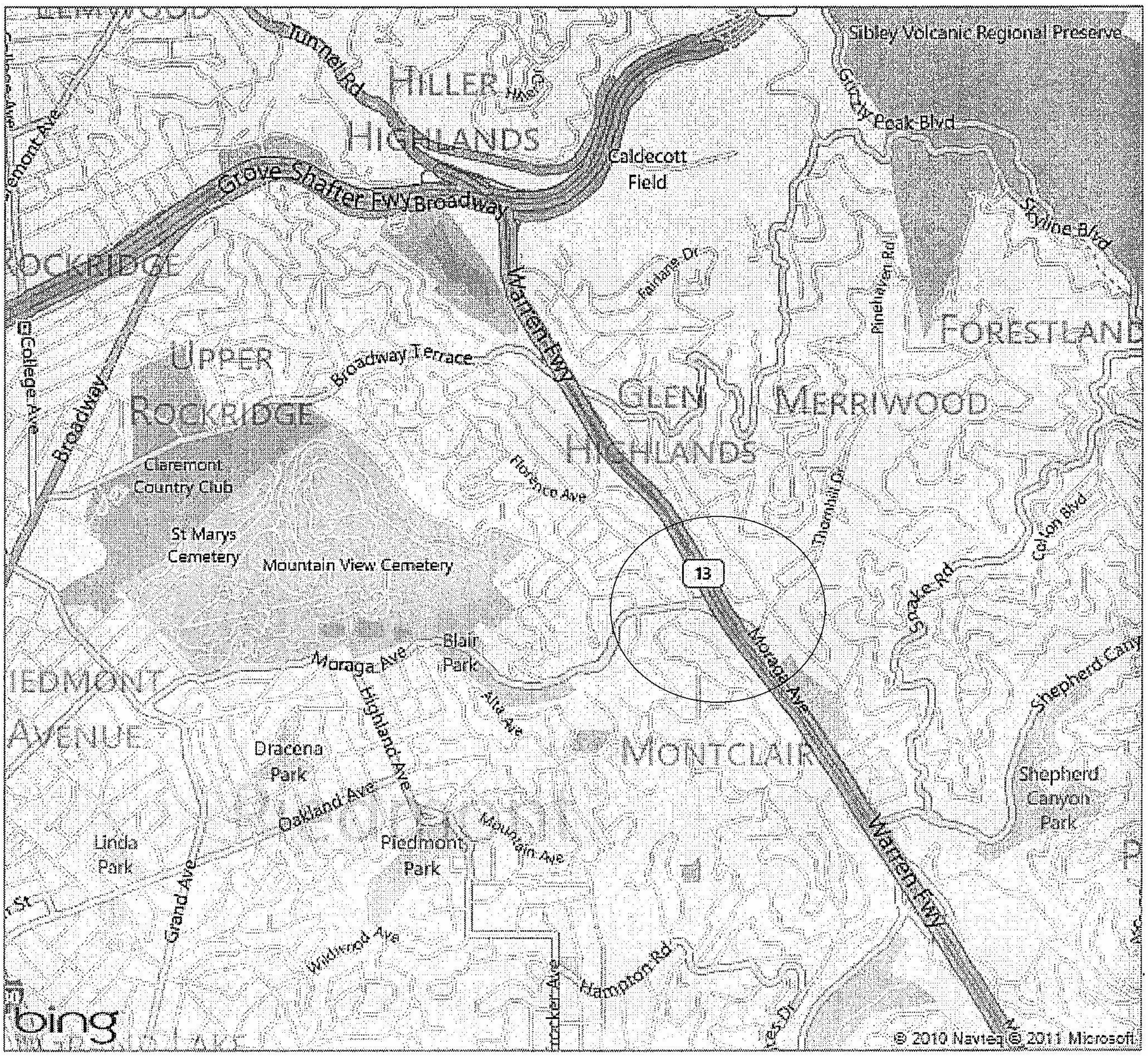


Attachments

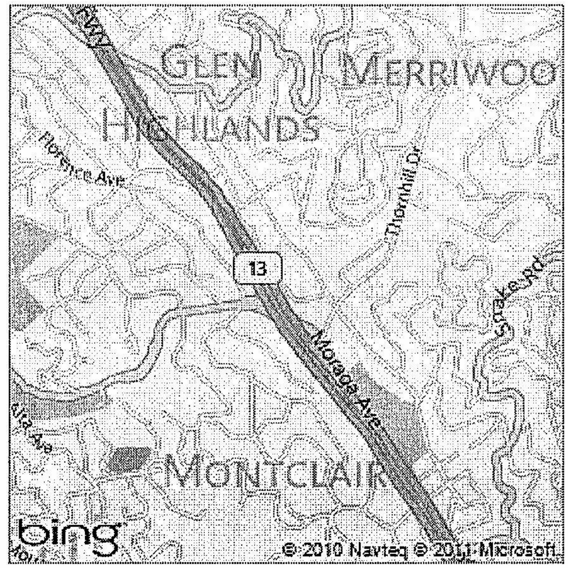
c: TPokrywka, MMomenzadeh, E☺rtega, Dwight_Manlulu, SRajendra, Melanie_Brent, RE_Pending_File@dot.ca.gov, John_Stayton. Tinu_Mishra

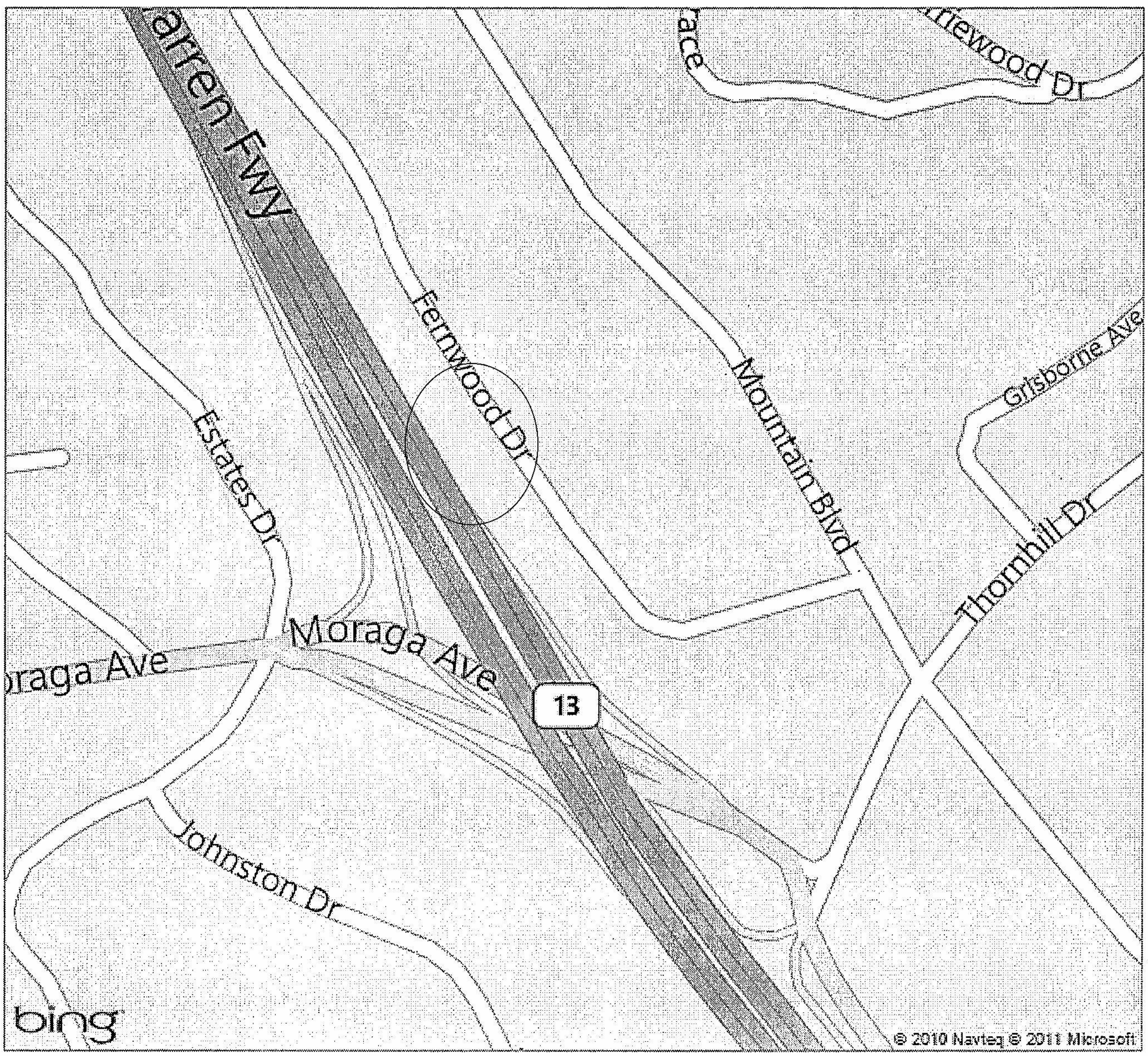
Eduardo ☺rtega/mm/FR Thornhill.docx

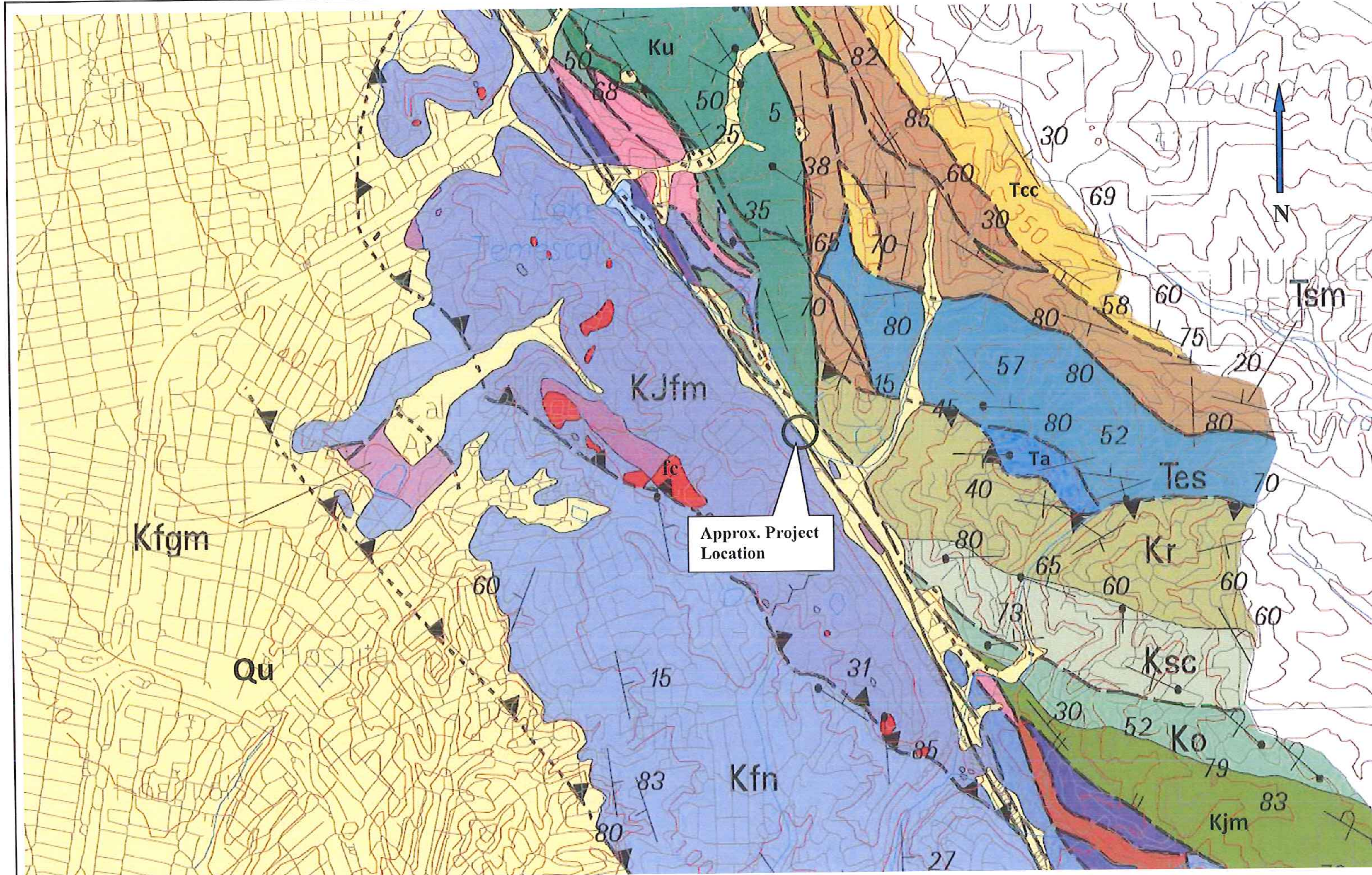
LOCATION MAP



VICINITY MAP







Geologic Units

Qu - Undivided surficial deposits.

Kfn - Novato Quarry terrane – sandstone and siltstone.

Kfgm- Diorite in Novato quarry terrane.

Kjfm - Melange terrane

Fc - chert

Sp - Serpentinite

Tcc - Claremont chert, sandstone interbeds.

Tsm - Unnamed sandstone and green mudstone.

Tes - Unnamedsandstone and green mudstone.

Ku - Unnamed sandstone and shale.

Kr - Redwood Canyon Formation sandstone.

Ksc - Shepherd Creek Formation sandstone.

Ko - Oakland Formation Conglomerate and shale.

Kjm - Joaquin Miller Formation fine sandstone and shale.

Jsv - Keratophyre and Quartz Keratophyre.

SCALE 1: 37,500

MAP TAKEN FROM:
PRELIMINARY GEOLOGIC MAP EMPHASIZING BEDROCK FORMATION IN ALAMEDA, CALIFORNIA,USGS, OFR 96-252

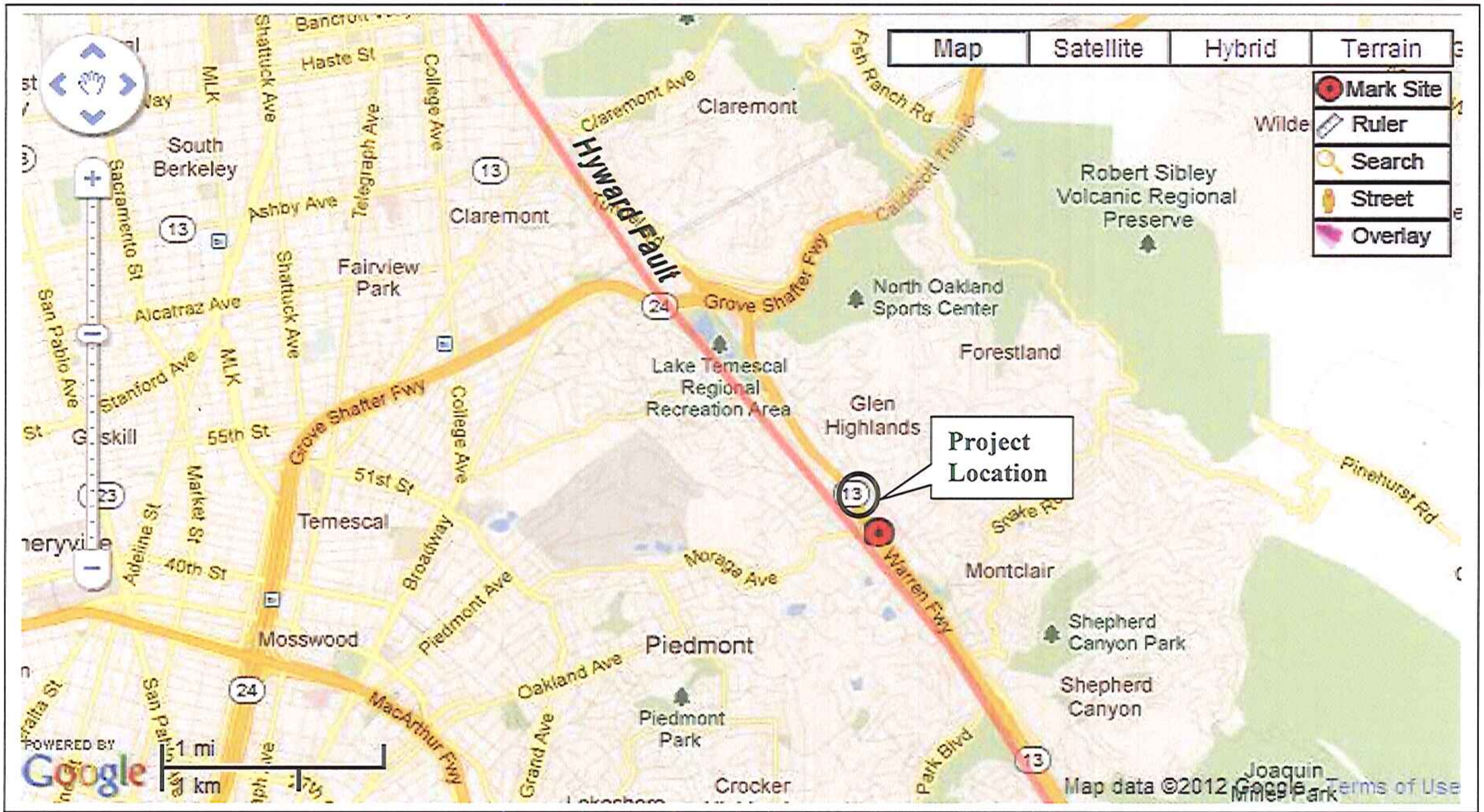
Caltrans

Engineering Service Center
DIVISION OF ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SERVICES
GEOTECHNICAL DESIGN BRANCH (WEST) – BRANCH B

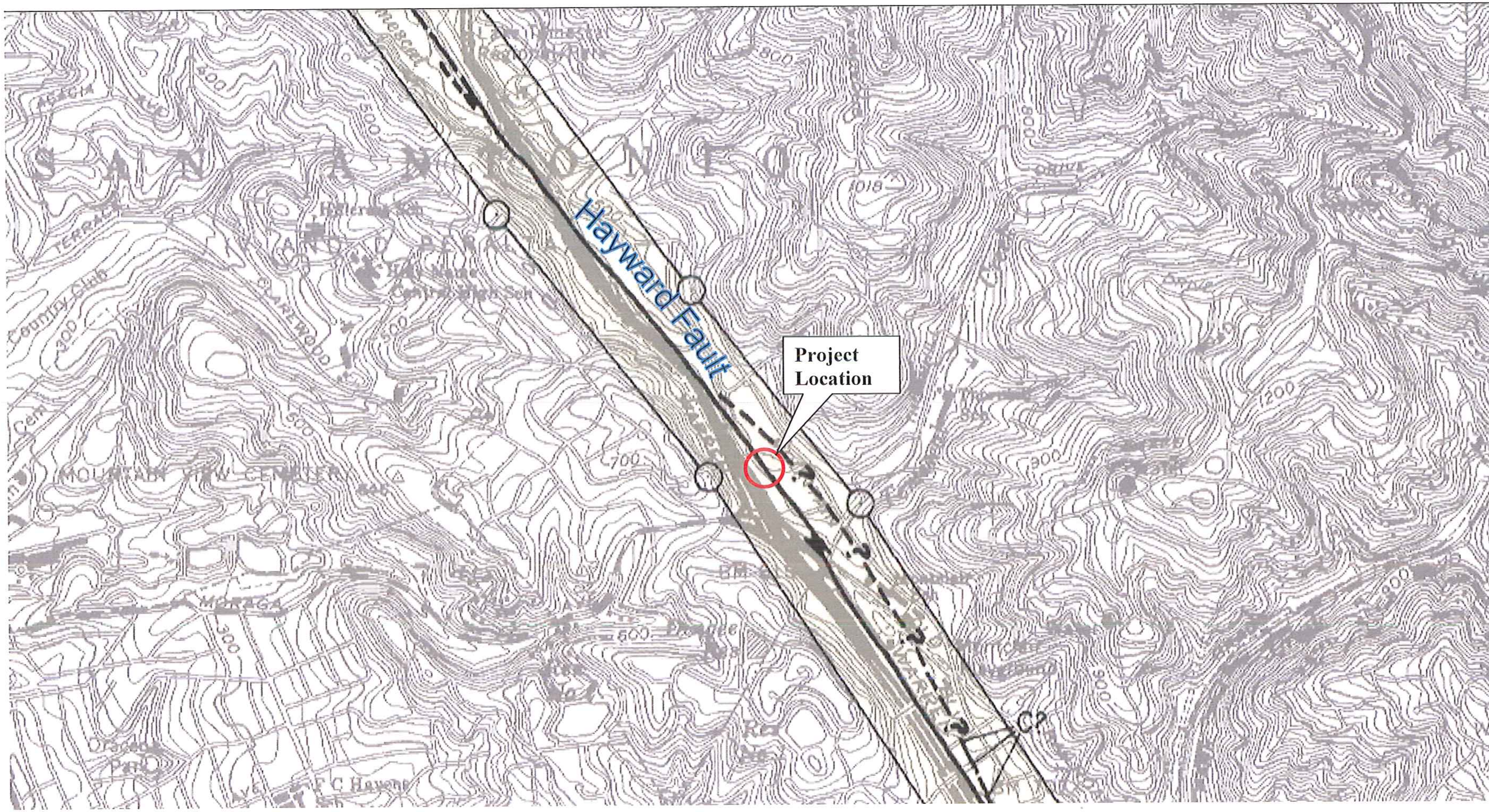
Geology MAP

04-ALA-13	EFIS 0400020868
PM 8.3	April 2012

FIGURE 2



MAP TAKEN FROM: USGS MARE ISLAND QUADRANGLE, 1980 SCALE Not to Scale	Caltrans	**Engineering Service Center** DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL SERVICES GEOTECHNICAL DESIGN BRANCH (WEST) – BRANCH B	**FAULT MAP** **04-ALA-13** **EFIS 0400020868** **PM 8.3** **April 2012** FIGURE ~~3~~ ~~XX~~ 1



MAP TAKEN FROM:
CGS, DECEMBER 2010, ALQUIST- PRIOLO EARTHQUAKE FAULT ZONE.

Scale

Not to Scale

Caltrans

Engineering Service Center
DIVISION OF ENGINEERING SERVICES
OFFICE OF GEOTECHNICAL SERVICES
GEOTECHNICAL DESIGN BRANCH (WEST) – BRANCH B

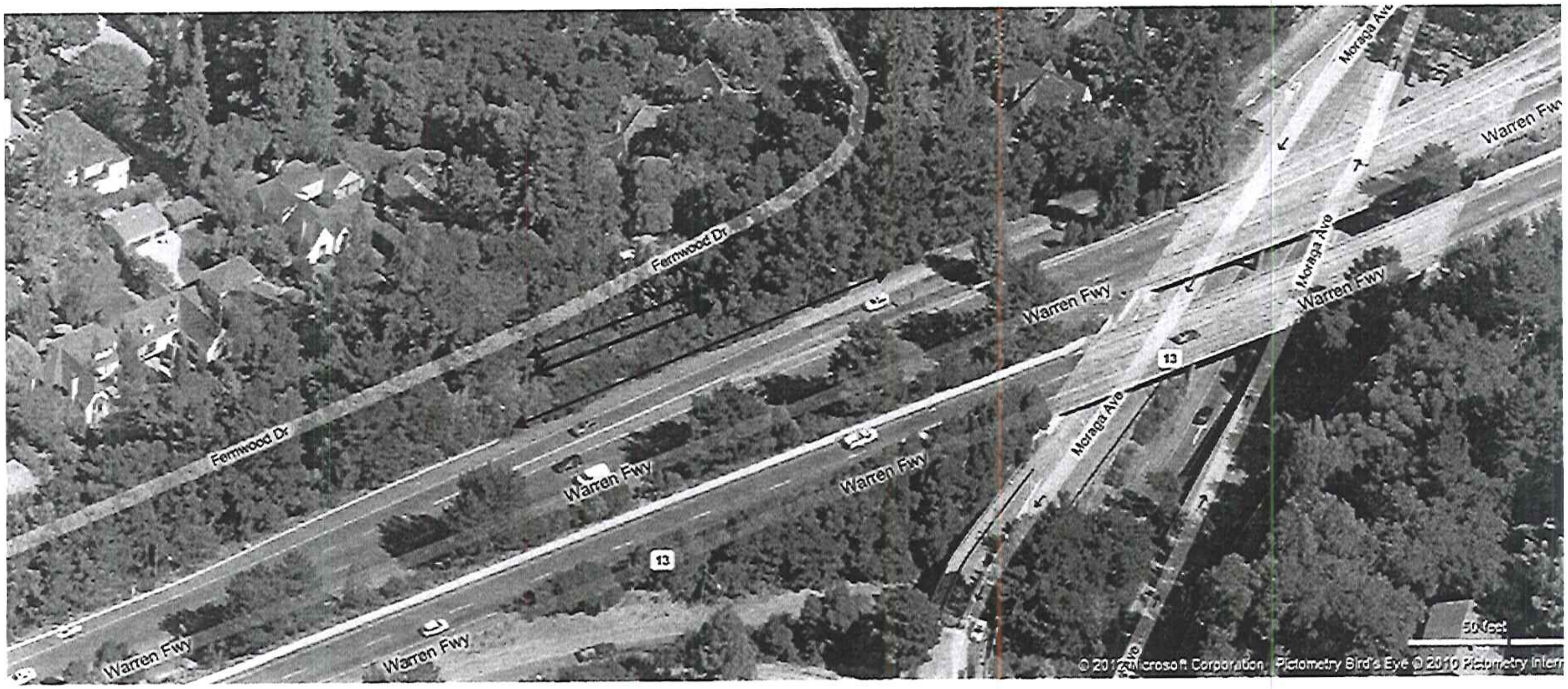
ALQUIST-PRIOLO **FAULT MAP**

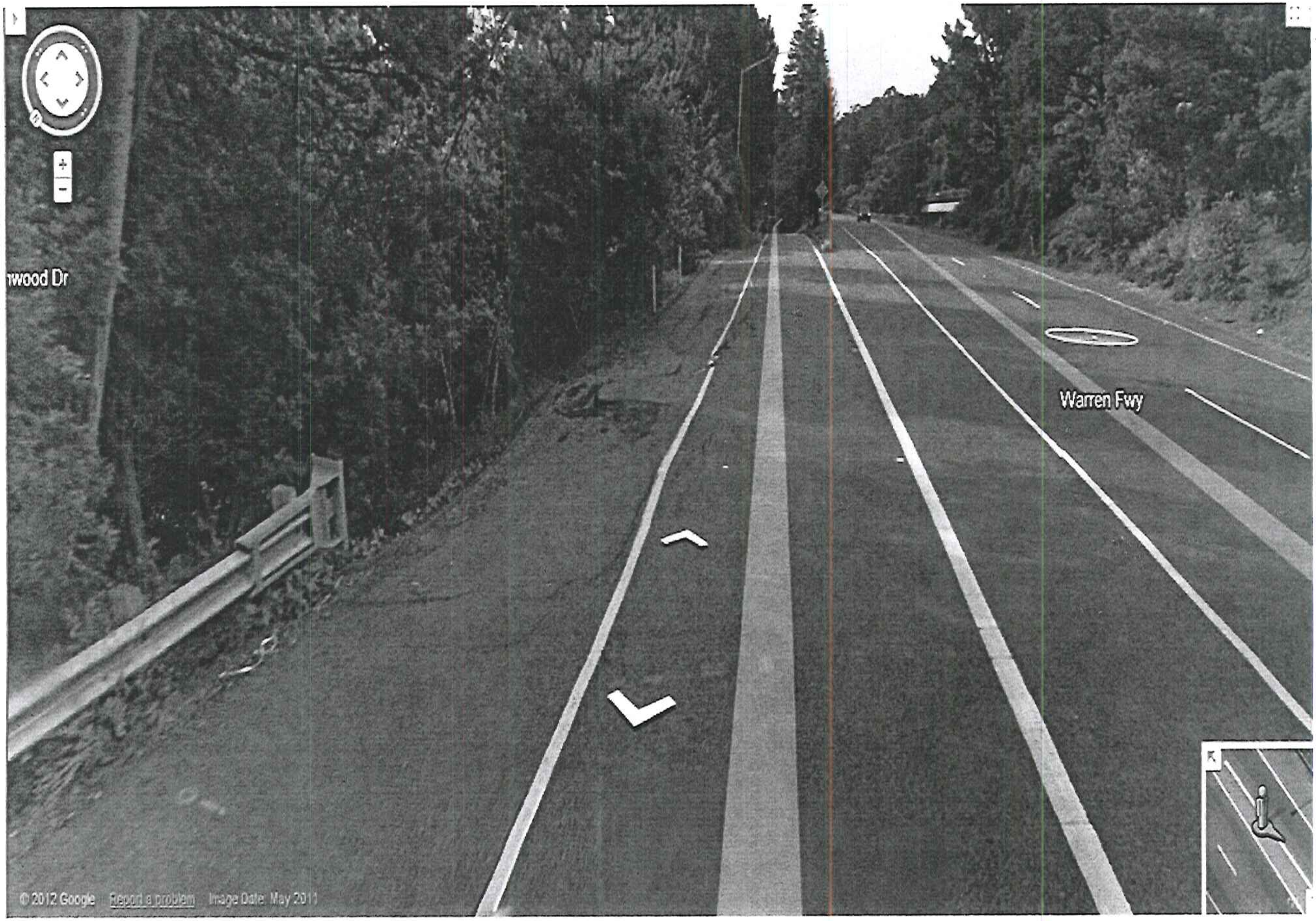
04-ALA-13 **EFIS 0400020868**

PM 8.3 **April 2012**

FIGURE 3

Location of the Project




nwood Dr
Warren Fwy
© 2012 Google Report a problem Image Date: May 2011


1573 Fernwood Drive, Oakland, California, United States
Address is approximate
Fernwood Dr
© 2012 Google Report a problem Image Date: May 2011

Credited to:

Jeffrey Monroe
OGDW
Orinda Field Station

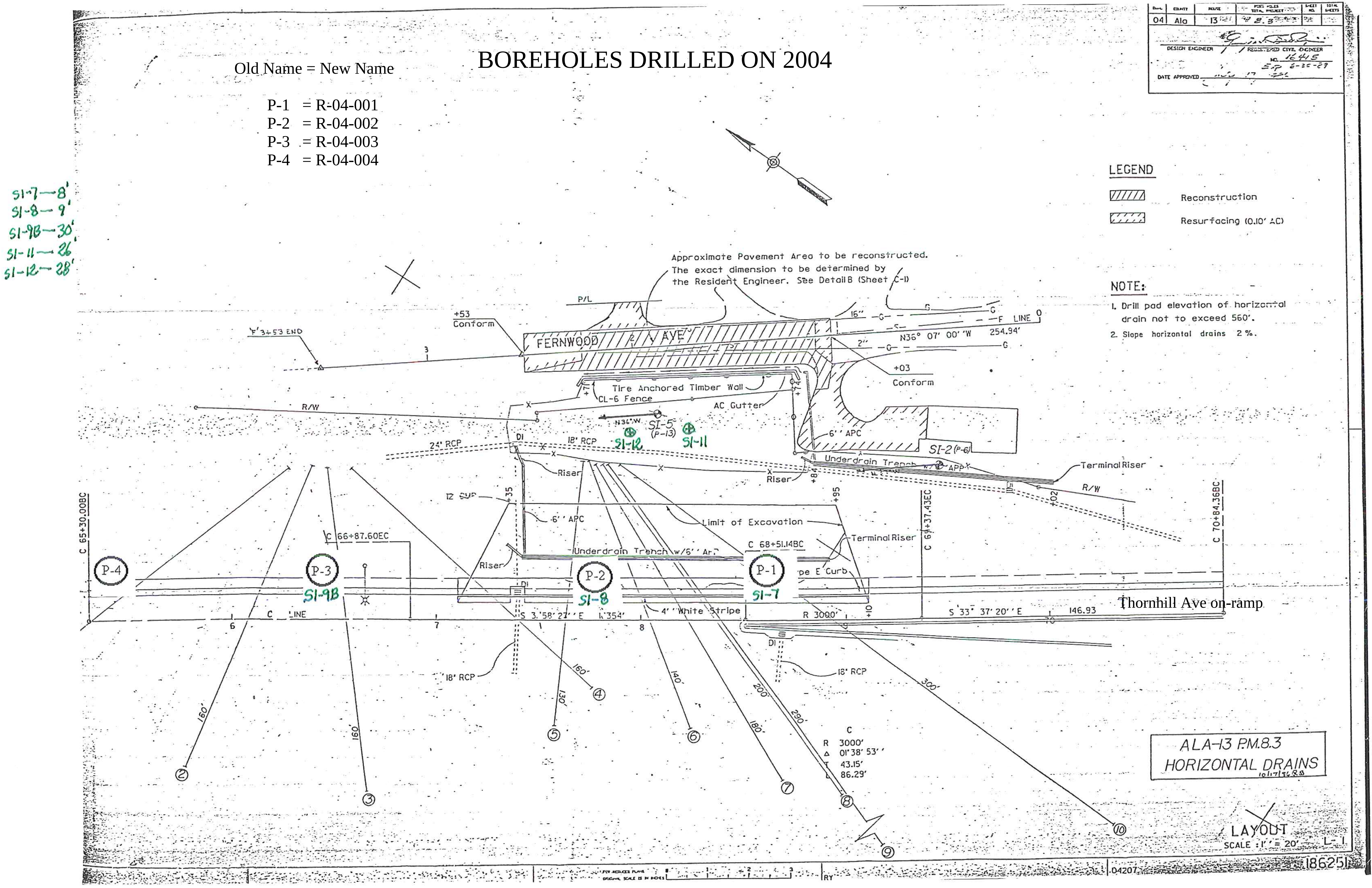
SITE MAP

Credited to: Jeffrey Monroe - OGDW - Orinda Field Station

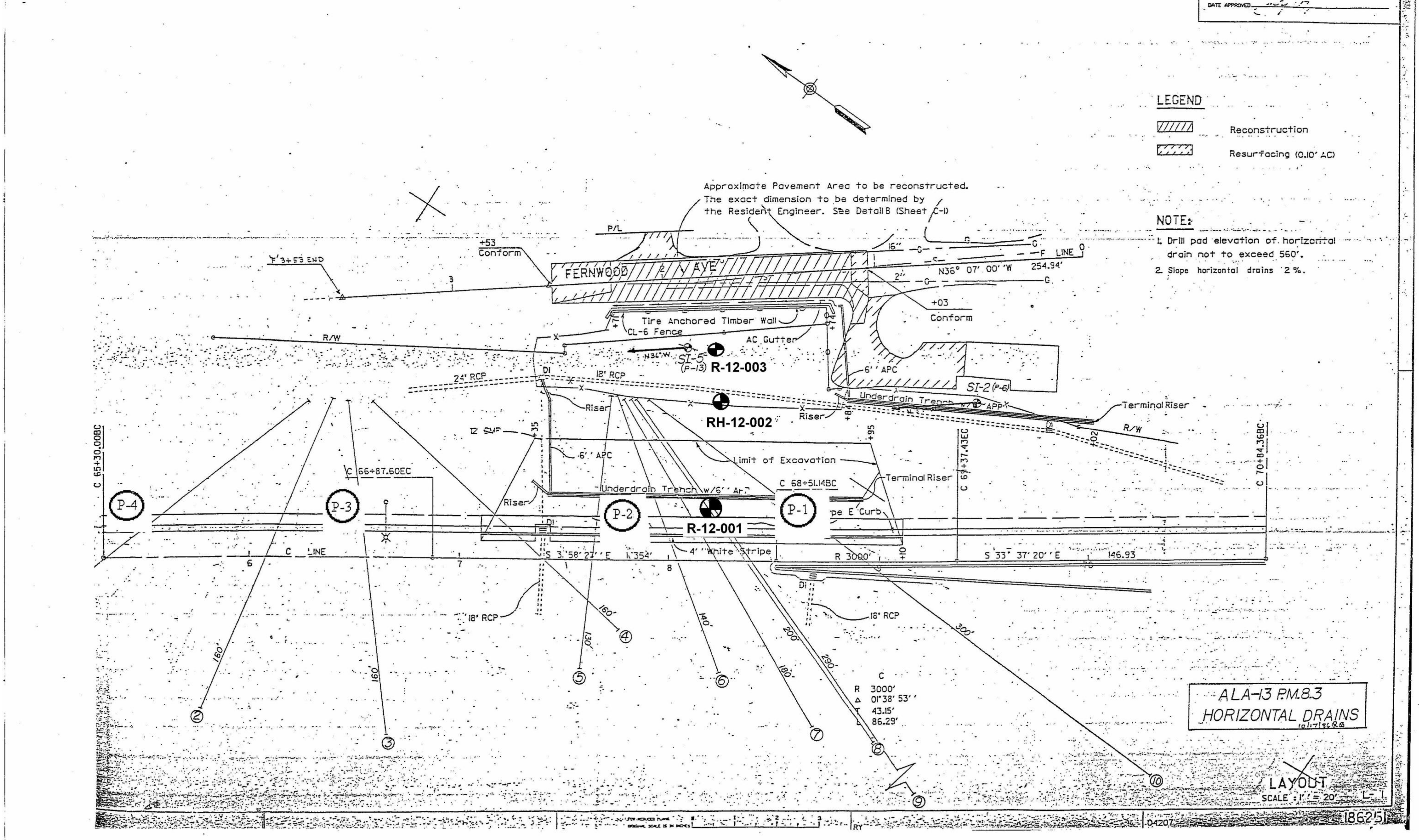
BOREHOLES DRILLED ON 2004

Old Name = New Name

P-1 = R-04-001
P-2 = R-04-002
P-3 = R-04-003
P-4 = R-04-004

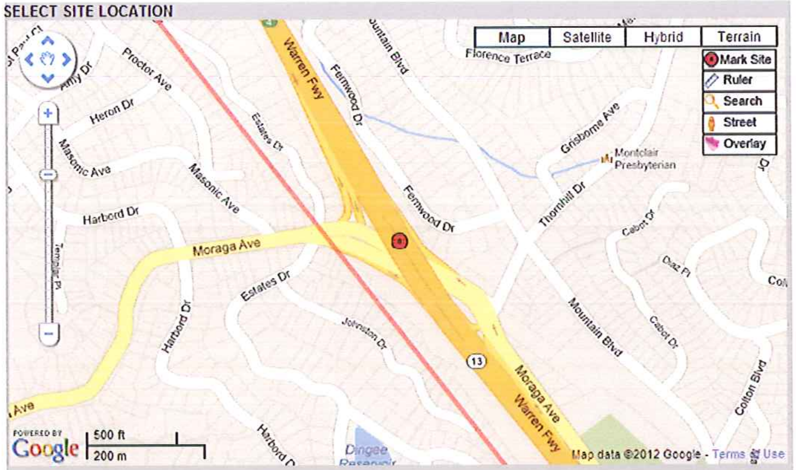


BOREHOLES DRLLED ON 2012



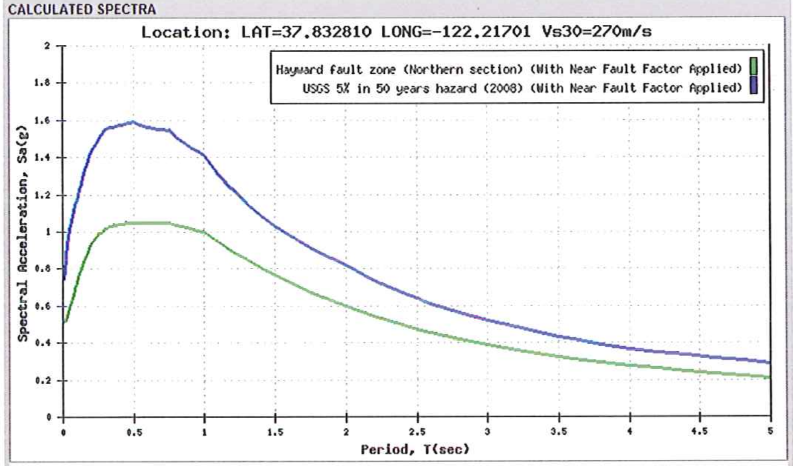
SEISMIC

SELECT SITE LOCATION



Latitude: 37.83281 Longitude: -122.21701 Vs30: 270 m/s Calculate

CALCULATED SPECTRA



View Tabular Data | Show Envelope Only | Hide Near Fault Adjustment | Axis Scale

Printer Friendly View | Show Basin Factor

Apply Near Fault Adjustment To:

NOTE: Caltrans SDC requires application of a Near Fault Adjustment factor for sites less than 25 km (Rrup) from the causative fault

☑ Deterministic Spectrum Using

0.05 Km Hayward fault zone (Northern section)

☑ Probabilistic Spectrum Using

0.05 Km (Recommend Performing Deaggregation To Verify)

◉ Show Spectrum with Adjustment Only

○ Show Spectrum with and without near fault Adjustment

OK

SITE DATA

Shear Wave Velocity, Vs_{30}:	270 m/s
Latitude:	37.832810
Longitude:	-122.217010
Depth to Vs = 1.0 km/s:	327 m
Depth to Vs = 2.5 km/s:	2.00 km

DETERMINISTIC

Hayward fault zone (Northern section)

Fault ID:	353
Maximum Magnitude (MMax):	7.3
Fault Type:	RLSS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	12.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	0.05 km
Rjb:	0.05 km
Rx:	0.05 km
Fnorm:	0
Frev:	0

Period	SA (Base Spectrum)	Basin Factor	Near Fault Factor (Applied)	SA (Final Spectrum)
0.01	0.513	1.000	1.000	0.513
0.02	0.521	1.000	1.000	0.521
0.022	0.526	1.000	1.000	0.526
0.025	0.533	1.000	1.000	0.533
0.029	0.541	1.000	1.000	0.541
0.03	0.543	1.000	1.000	0.543
0.032	0.549	1.000	1.000	0.549
0.035	0.557	1.000	1.000	0.557
0.036	0.559	1.000	1.000	0.559
0.04	0.569	1.000	1.000	0.569
0.042	0.574	1.000	1.000	0.574
0.044	0.580	1.000	1.000	0.580
0.045	0.582	1.000	1.000	0.582
0.046	0.585	1.000	1.000	0.585
0.048	0.591	1.000	1.000	0.591

0.05	0.596	1.000	1.000	0.596
0.055	0.606	1.000	1.000	0.606
0.06	0.617	1.000	1.000	0.617
0.065	0.629	1.000	1.000	0.629
0.067	0.634	1.000	1.000	0.634
0.07	0.641	1.000	1.000	0.641
0.075	0.653	1.000	1.000	0.653
0.08	0.667	1.000	1.000	0.667
0.085	0.681	1.000	1.000	0.681
0.09	0.695	1.000	1.000	0.695
0.095	0.709	1.000	1.000	0.709
0.1	0.723	1.000	1.000	0.723
0.11	0.750	1.000	1.000	0.750
0.12	0.775	1.000	1.000	0.775
0.13	0.798	1.000	1.000	0.798
0.133	0.804	1.000	1.000	0.804
0.14	0.818	1.000	1.000	0.818
0.15	0.836	1.000	1.000	0.836
0.16	0.857	1.000	1.000	0.857
0.17	0.876	1.000	1.000	0.876
0.18	0.894	1.000	1.000	0.894
0.19	0.911	1.000	1.000	0.911
0.2	0.927	1.000	1.000	0.927
0.22	0.952	1.000	1.000	0.952
0.24	0.973	1.000	1.000	0.973
0.25	0.983	1.000	1.000	0.983
0.26	0.989	1.000	1.000	0.989
0.28	1.003	1.000	1.000	1.003
0.29	1.007	1.000	1.000	1.007
0.3	1.013	1.000	1.000	1.013
0.32	1.022	1.000	1.000	1.022
0.34	1.029	1.000	1.000	1.029
0.35	1.031	1.000	1.000	1.031
0.36	1.034	1.000	1.000	1.034
0.38	1.037	1.000	1.000	1.037
0.4	1.039	1.000	1.000	1.039
0.42	1.042	1.000	1.000	1.042
0.44	1.044	1.000	1.000	1.044
0.45	1.045	1.000	1.000	1.045
0.46	1.046	1.000	1.000	1.046
0.48	1.047	1.000	1.000	1.047
0.5	1.048	1.000	1.000	1.048
0.55	1.027	1.000	1.020	1.047
0.6	1.007	1.000	1.040	1.048
0.65	0.989	1.000	1.060	1.048
0.667	0.982	1.000	1.067	1.048

0.7	0.970	1.000	1.080	1.048
0.75	0.953	1.000	1.100	1.048
0.8	0.927	1.000	1.120	1.038
0.85	0.902	1.000	1.140	1.028
0.9	0.877	1.000	1.160	1.018
0.95	0.855	1.000	1.180	1.009
1	0.833	1.000	1.200	0.999
1.1	0.787	1.000	1.200	0.945
1.2	0.746	1.000	1.200	0.895
1.3	0.708	1.000	1.200	0.849
1.4	0.673	1.000	1.200	0.808
1.5	0.640	1.000	1.200	0.768
1.6	0.607	1.000	1.200	0.728
1.7	0.576	1.000	1.200	0.691
1.8	0.548	1.000	1.200	0.658
1.9	0.523	1.000	1.200	0.628
2	0.501	1.000	1.200	0.601
2.2	0.454	1.000	1.200	0.545
2.4	0.414	1.000	1.200	0.497
2.5	0.397	1.000	1.200	0.476
2.6	0.380	1.000	1.200	0.456
2.8	0.350	1.000	1.200	0.420
3	0.324	1.000	1.200	0.389
3.2	0.301	1.000	1.200	0.361
3.4	0.280	1.000	1.200	0.336
3.5	0.271	1.000	1.200	0.325
3.6	0.262	1.000	1.200	0.314
3.8	0.245	1.000	1.200	0.294
4	0.230	1.000	1.200	0.276
4.2	0.218	1.000	1.200	0.261
4.4	0.206	1.000	1.200	0.247
4.6	0.195	1.000	1.200	0.234
4.8	0.185	1.000	1.200	0.222
5	0.176	1.000	1.200	0.211

To use above data in Excel, copy/paste:

```
0.01    0.513    1.000    1.000    0.513
0.02    0.521    1.000    1.000    0.521
```

PROBABILISTIC

Probabilistic Model
USGS Seismic Hazard Map(2008) 975 Year Return Period

Period	**SA (Base Spectrum)**	**Basin Factor**	**Near Fault Factor (Applied)**	**SA (Final Spectrum)**
0.01	0.742	1.000	1.000	0.742

0.02	0.851	1.000	1.000	0.851
0.022	0.867	1.000	1.000	0.867
0.025	0.889	1.000	1.000	0.889
0.029	0.915	1.000	1.000	0.915
0.03	0.921	1.000	1.000	0.921
0.032	0.933	1.000	1.000	0.933
0.035	0.950	1.000	1.000	0.950
0.036	0.955	1.000	1.000	0.955
0.04	0.975	1.000	1.000	0.975
0.042	0.984	1.000	1.000	0.984
0.044	0.993	1.000	1.000	0.993
0.045	0.998	1.000	1.000	0.998
0.046	1.002	1.000	1.000	1.002
0.048	1.011	1.000	1.000	1.011
0.05	1.019	1.000	1.000	1.019
0.055	1.038	1.000	1.000	1.038
0.06	1.056	1.000	1.000	1.056
0.065	1.073	1.000	1.000	1.073
0.067	1.079	1.000	1.000	1.079
0.07	1.088	1.000	1.000	1.088
0.075	1.103	1.000	1.000	1.103
0.08	1.117	1.000	1.000	1.117
0.085	1.131	1.000	1.000	1.131
0.09	1.144	1.000	1.000	1.144
0.095	1.156	1.000	1.000	1.156
0.1	1.168	1.000	1.000	1.168
0.11	1.200	1.000	1.000	1.200
0.12	1.230	1.000	1.000	1.230
0.13	1.259	1.000	1.000	1.259
0.133	1.267	1.000	1.000	1.267
0.14	1.286	1.000	1.000	1.286
0.15	1.312	1.000	1.000	1.312
0.16	1.337	1.000	1.000	1.337
0.17	1.360	1.000	1.000	1.360
0.18	1.383	1.000	1.000	1.383
0.19	1.405	1.000	1.000	1.405
0.2	1.425	1.000	1.000	1.425
0.22	1.454	1.000	1.000	1.454
0.24	1.481	1.000	1.000	1.481
0.25	1.494	1.000	1.000	1.494
0.26	1.506	1.000	1.000	1.506
0.28	1.530	1.000	1.000	1.530
0.29	1.541	1.000	1.000	1.541
0.3	1.552	1.000	1.000	1.552
0.32	1.557	1.000	1.000	1.557
0.34	1.562	1.000	1.000	1.562

0.35	1.564	1.000	1.000	1.564
0.36	1.566	1.000	1.000	1.566
0.38	1.570	1.000	1.000	1.570
0.4	1.574	1.000	1.000	1.574
0.42	1.577	1.000	1.000	1.577
0.44	1.581	1.000	1.000	1.581
0.45	1.583	1.000	1.000	1.583
0.46	1.584	1.000	1.000	1.584
0.48	1.587	1.000	1.000	1.587
0.5	1.591	1.000	1.000	1.591
0.55	1.544	1.000	1.020	1.575
0.6	1.504	1.000	1.040	1.564
0.65	1.467	1.000	1.060	1.555
0.667	1.455	1.000	1.067	1.553
0.7	1.434	1.000	1.080	1.549
0.75	1.404	1.000	1.100	1.544
0.8	1.350	1.000	1.120	1.512
0.85	1.301	1.000	1.140	1.483
0.9	1.257	1.000	1.160	1.458
0.95	1.216	1.000	1.180	1.435
1	1.179	1.000	1.200	1.415
1.1	1.094	1.000	1.200	1.313
1.2	1.022	1.000	1.200	1.226
1.3	0.960	1.000	1.200	1.152
1.4	0.906	1.000	1.200	1.087
1.5	0.858	1.000	1.200	1.030
1.6	0.816	1.000	1.200	0.979
1.7	0.778	1.000	1.200	0.934
1.8	0.744	1.000	1.200	0.893
1.9	0.713	1.000	1.200	0.856
2	0.685	1.000	1.200	0.822
2.2	0.616	1.000	1.200	0.739
2.4	0.559	1.000	1.200	0.670
2.5	0.534	1.000	1.200	0.641
2.6	0.511	1.000	1.200	0.613
2.8	0.470	1.000	1.200	0.564
3	0.435	1.000	1.200	0.522
3.2	0.402	1.000	1.200	0.483
3.4	0.374	1.000	1.200	0.448
3.5	0.361	1.000	1.200	0.433
3.6	0.349	1.000	1.200	0.418
3.8	0.326	1.000	1.200	0.392
4	0.307	1.000	1.200	0.368
4.2	0.291	1.000	1.200	0.349
4.4	0.277	1.000	1.200	0.332
4.6	0.264	1.000	1.200	0.317

4.8	0.252	1.000	1.200	0.303
5	0.242	1.000	1.200	0.290

To use above data in Excel, copy/paste:

```
0.01    0.742    1.000    1.000    0.742
0.02    0.851    1.000    1.000    0.851
```

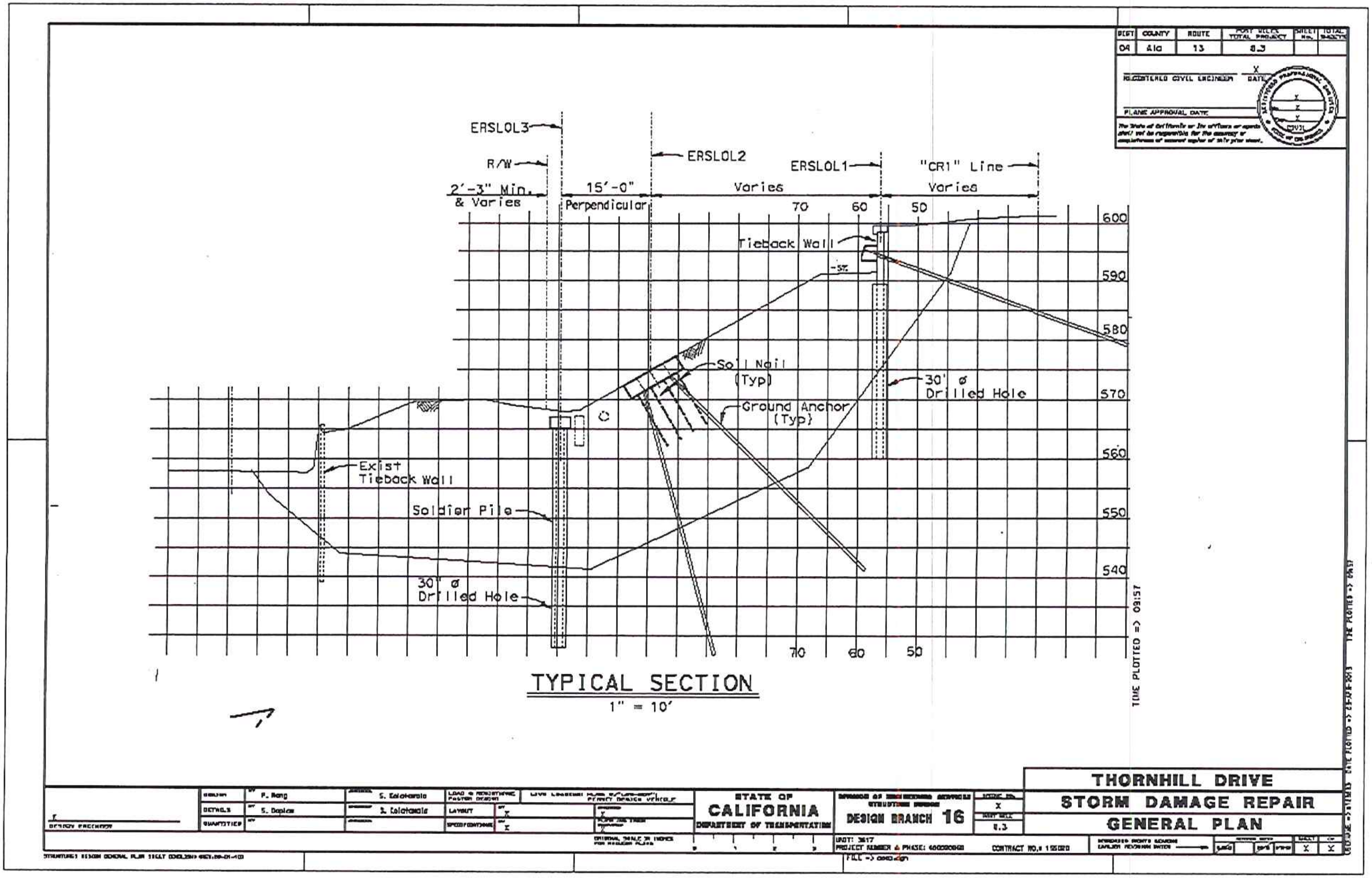
Envelope Data

Period	SA
0.01	0.742
0.02	0.851
0.022	0.867
0.025	0.889
0.029	0.915
0.03	0.921
0.032	0.933
0.035	0.950
0.036	0.955
0.04	0.975
0.042	0.984
0.044	0.993
0.045	0.998
0.046	1.002
0.048	1.011
0.05	1.019
0.055	1.038
0.06	1.056
0.065	1.073
0.067	1.079
0.07	1.088
0.075	1.103
0.08	1.117
0.085	1.131
0.09	1.144
0.095	1.156
0.1	1.168
0.11	1.200
0.12	1.230
0.13	1.259
0.133	1.267
0.14	1.286
0.15	1.312
0.16	1.337
0.17	1.360
0.18	1.383

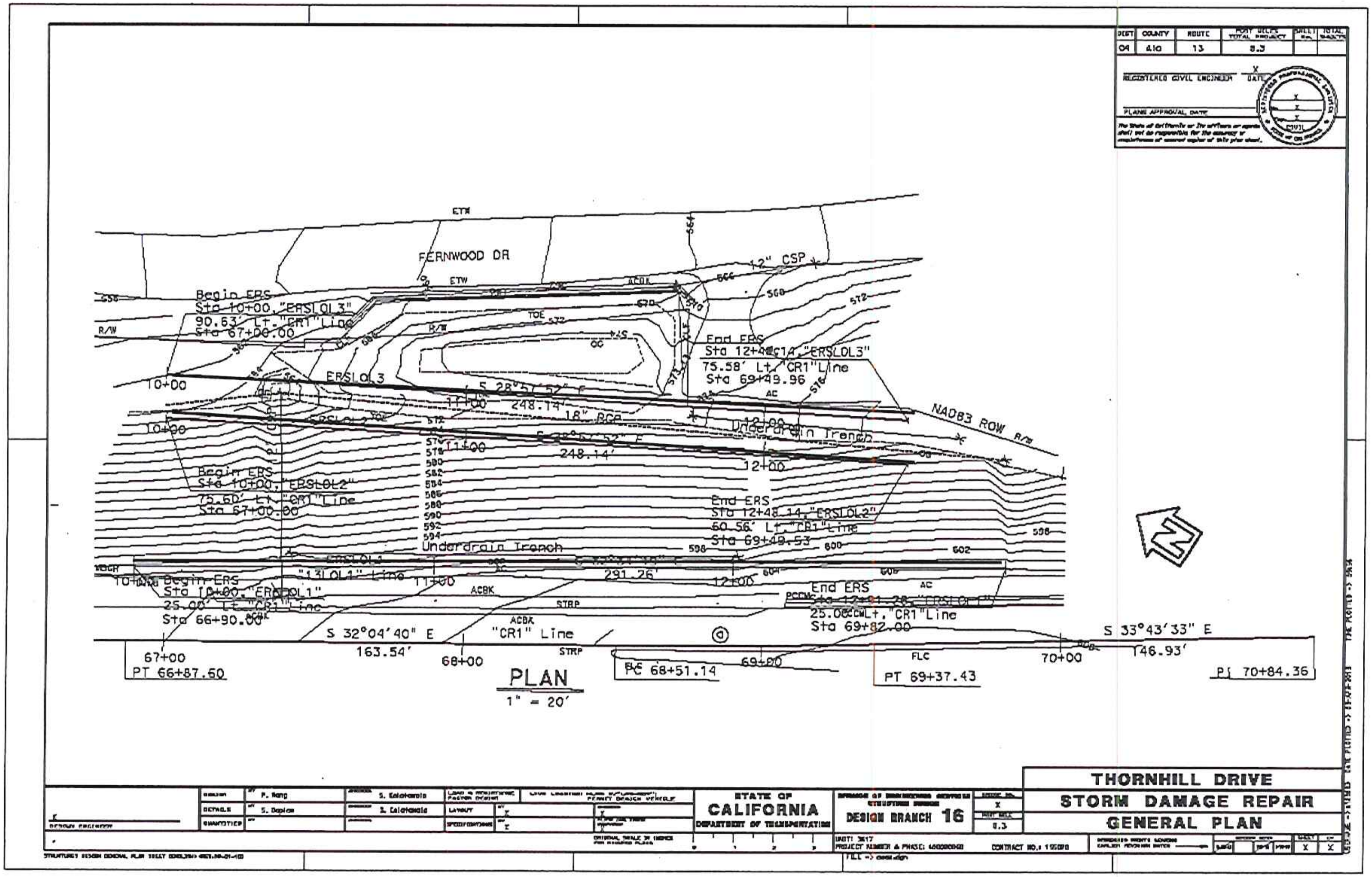
0.19	1.405
0.2	1.425
0.22	1.454
0.24	1.481
0.25	1.494
0.26	1.506
0.28	1.530
0.29	1.541
0.3	1.552
0.32	1.557
0.34	1.562
0.35	1.564
0.36	1.566
0.38	1.570
0.4	1.574
0.42	1.577
0.44	1.581
0.45	1.583
0.46	1.584
0.48	1.587
0.5	1.591
0.55	1.575
0.6	1.564
0.65	1.555
0.667	1.553
0.7	1.549
0.75	1.544
0.8	1.512
0.85	1.483
0.9	1.458
0.95	1.435
1	1.415
1.1	1.313
1.2	1.226
1.3	1.152
1.4	1.087
1.5	1.030
1.6	0.979
1.7	0.934
1.8	0.893
1.9	0.856
2	0.822
2.2	0.739
2.4	0.670
2.5	0.641
2.6	0.613

2.8	0.564
3	0.522
3.2	0.483
3.4	0.448
3.5	0.433
3.6	0.418
3.8	0.392
4	0.368
4.2	0.349
4.4	0.332
4.6	0.317
4.8	0.303
5	0.290
To use above data in Excel, copy/paste:	0.01 0.742 0.02 0.851

General Plan – Typical Section



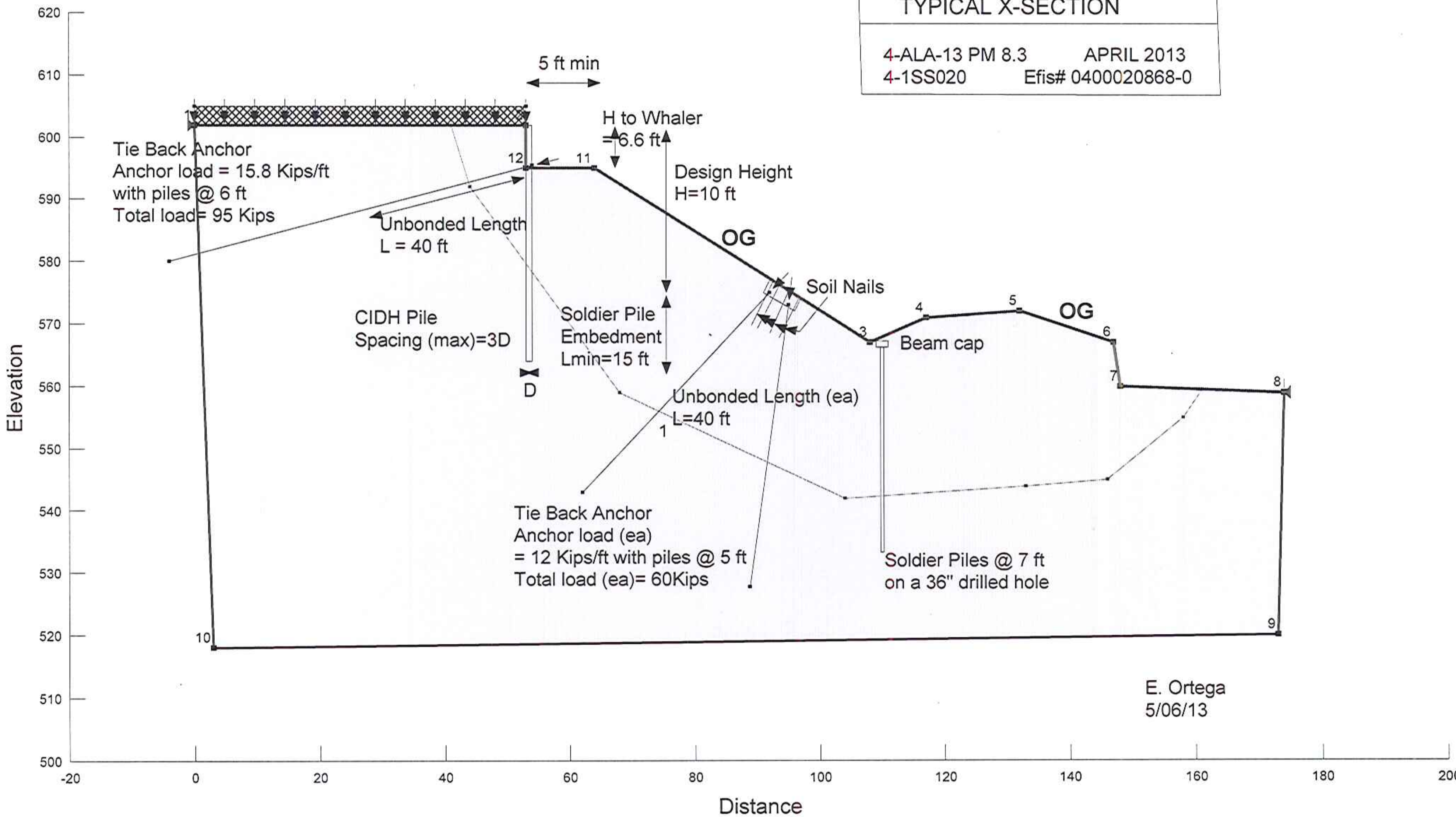
General Plan – Plan View



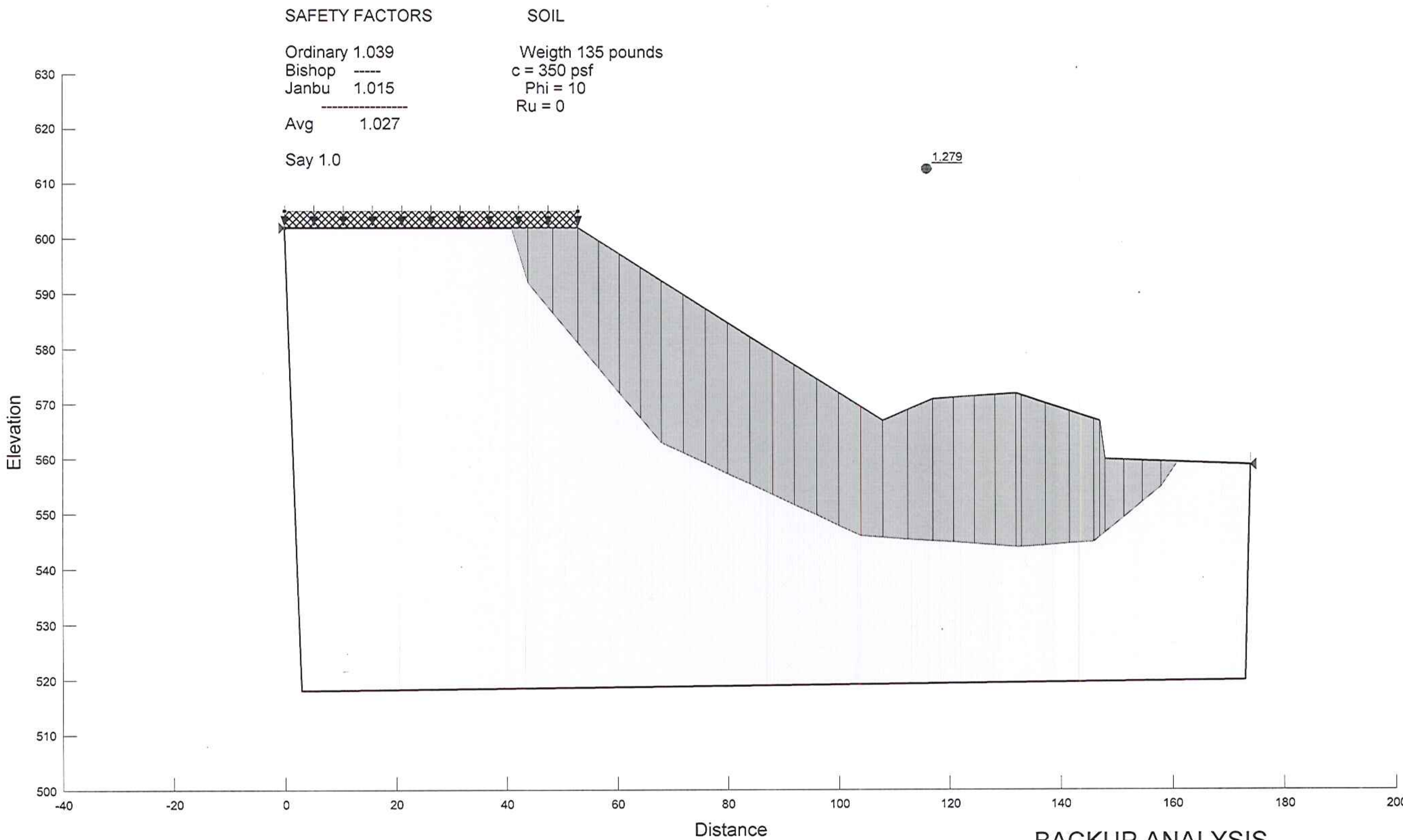
Office of Geotechnical Design - West
Geotechnical Services
Division of Engineering Services

STORM DAMAGE REPAIR
TYPICAL X-SECTION

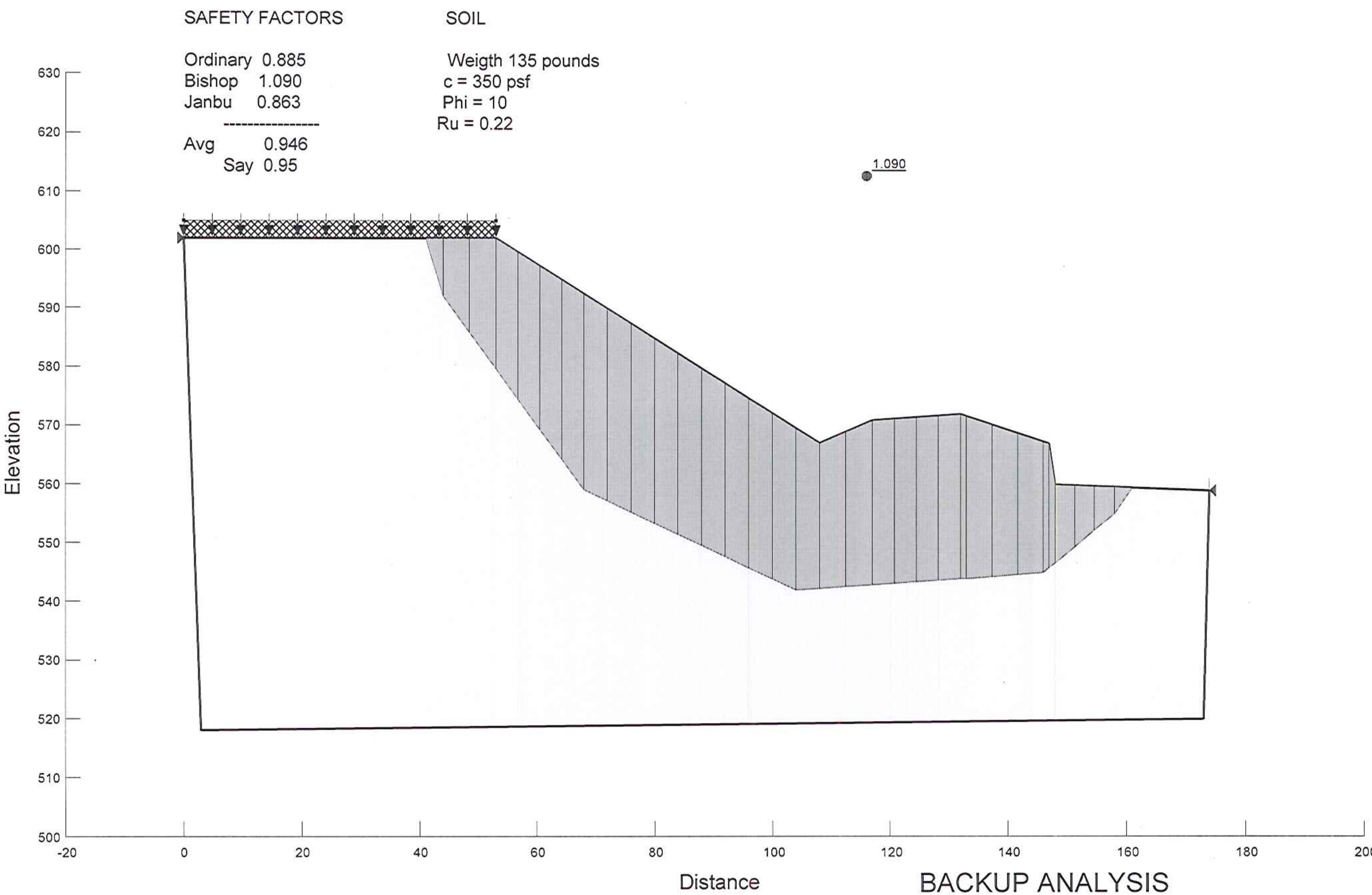
4-ALA-13 PM 8.3 APRIL 2013
4-1SS020 Efis# 0400020868-0



E. Ortega
5/06/13



BACKUP ANALYSIS
DRY CONDITION



BACKUP ANALYSIS
WET CONDITION

SAFETY FACTORS

Ordinary 1.293
Bishop 1.536
Janbu 1.184

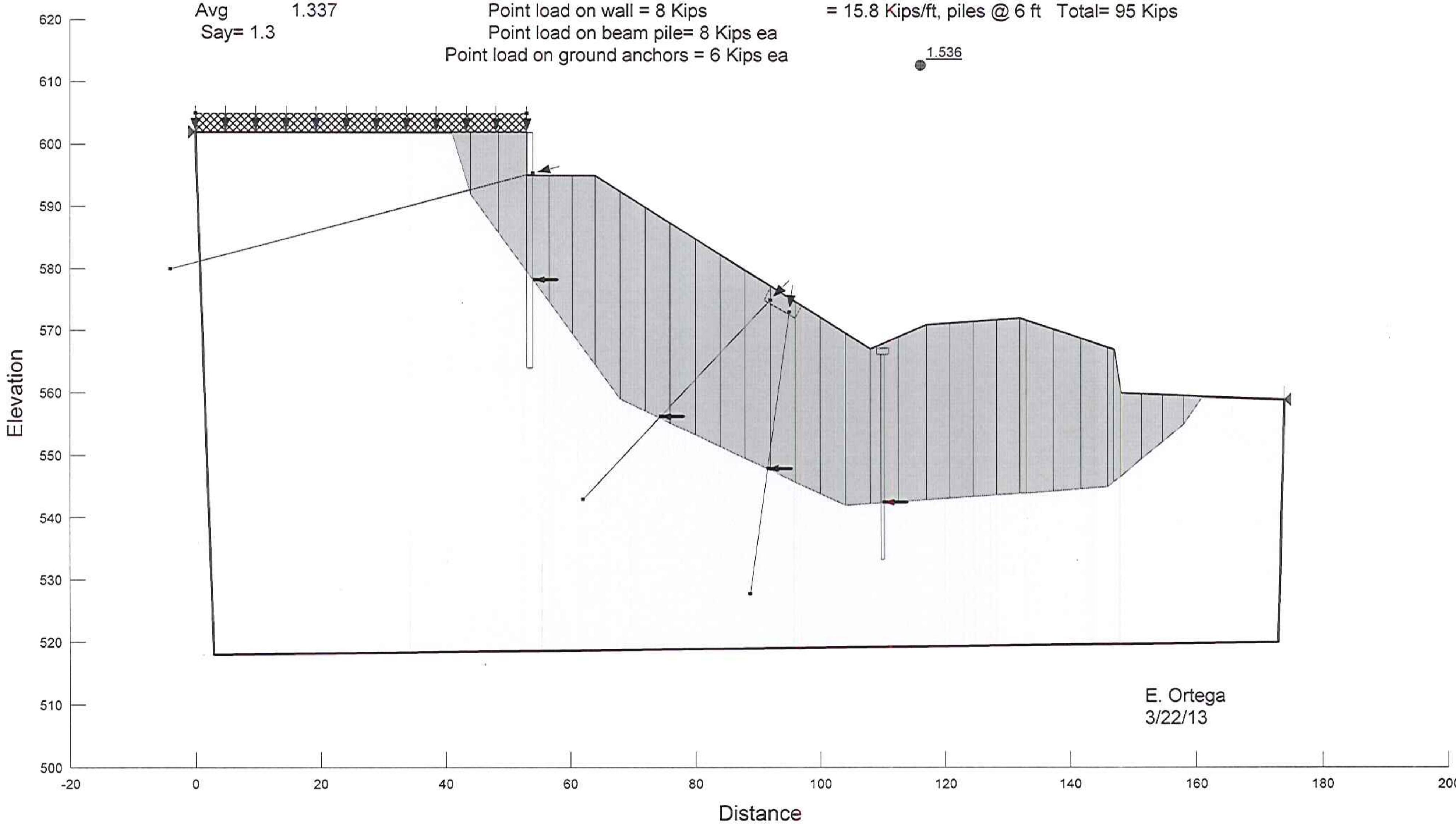
Avg 1.337
Say= 1.3

SOIL

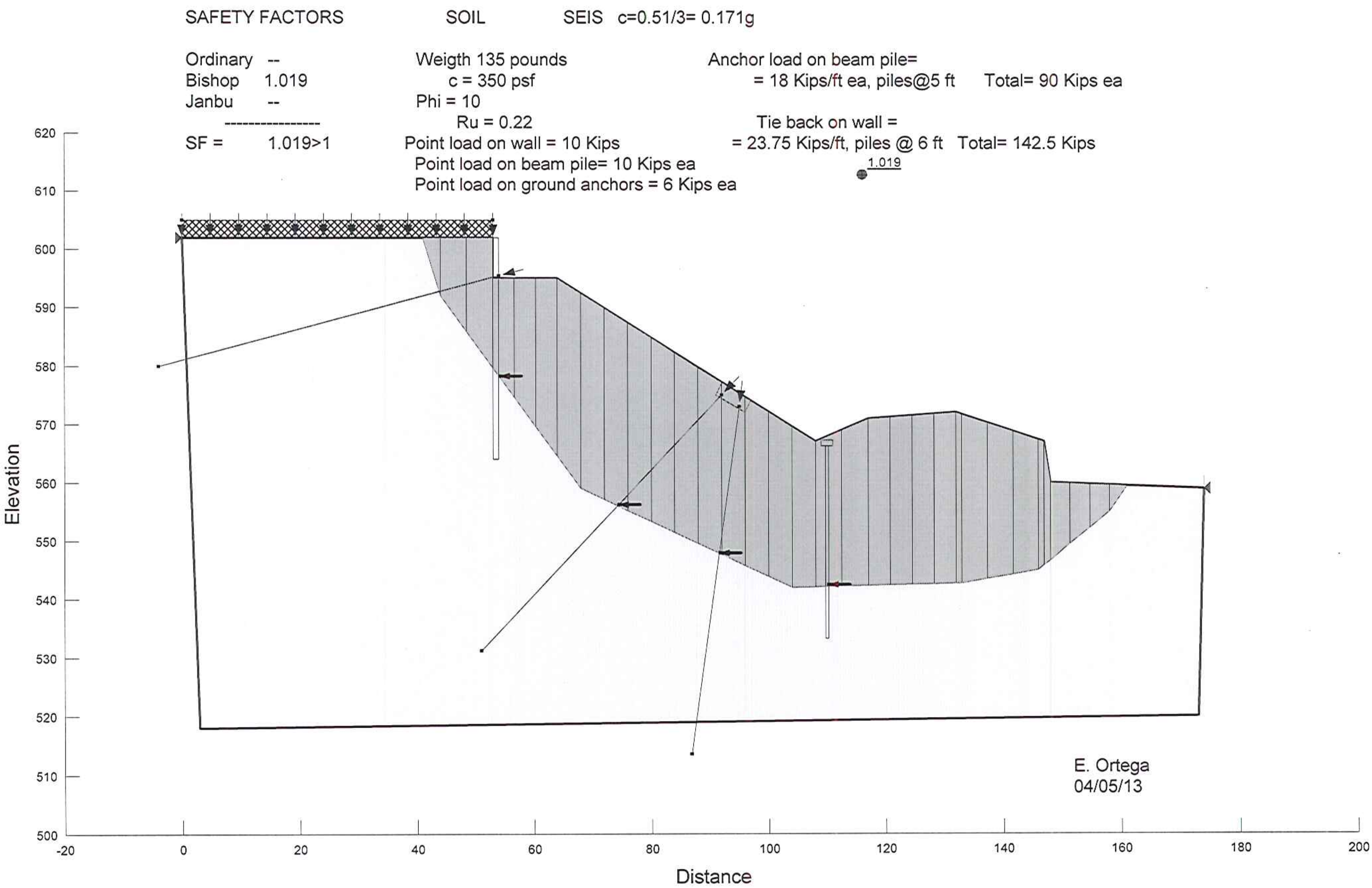
Weigth 135 pounds
c = 350 psf
Phi = 10
Ru = 0.22
Point load on wall = 8 Kips
Point load on beam pile= 8 Kips ea
Point load on ground anchors = 6 Kips ea

Anchor load on beam pile=
= 12 Kips/ft ea, piles@5 ft Total= 60 Kips ea

Tie back on wall =
= 15.8 Kips/ft, piles @ 6 ft Total= 95 Kips



E. Ortega
3/22/13



SAFETY FACTORS
Ordinary --
Bishop 1.019
Janbu --
SF = 1.019>1
SOIL
Weigth 135 pounds
c = 350 psf
Phi = 10
Ru = 0.22
Point load on wall = 10 Kips
Point load on beam pile= 10 Kips ea
Point load on ground anchors = 6 Kips ea
SEIS c=0.51/3= 0.171g
Anchor load on beam pile=
= 18 Kips/ft ea, piles@5 ft Total= 90 Kips ea
Tie back on wall =
= 23.75 Kips/ft, piles @ 6 ft Total= 142.5 Kips
1.019
Elevation
Distance
E. Ortega
04/05/13

5.5.5.7 Lateral Earth Pressures for Anchored Walls

For anchored walls restrained by tie rods and structural anchors, the lateral earth pressure acting on the wall may be determined in accordance with Article 5.5.5.6.

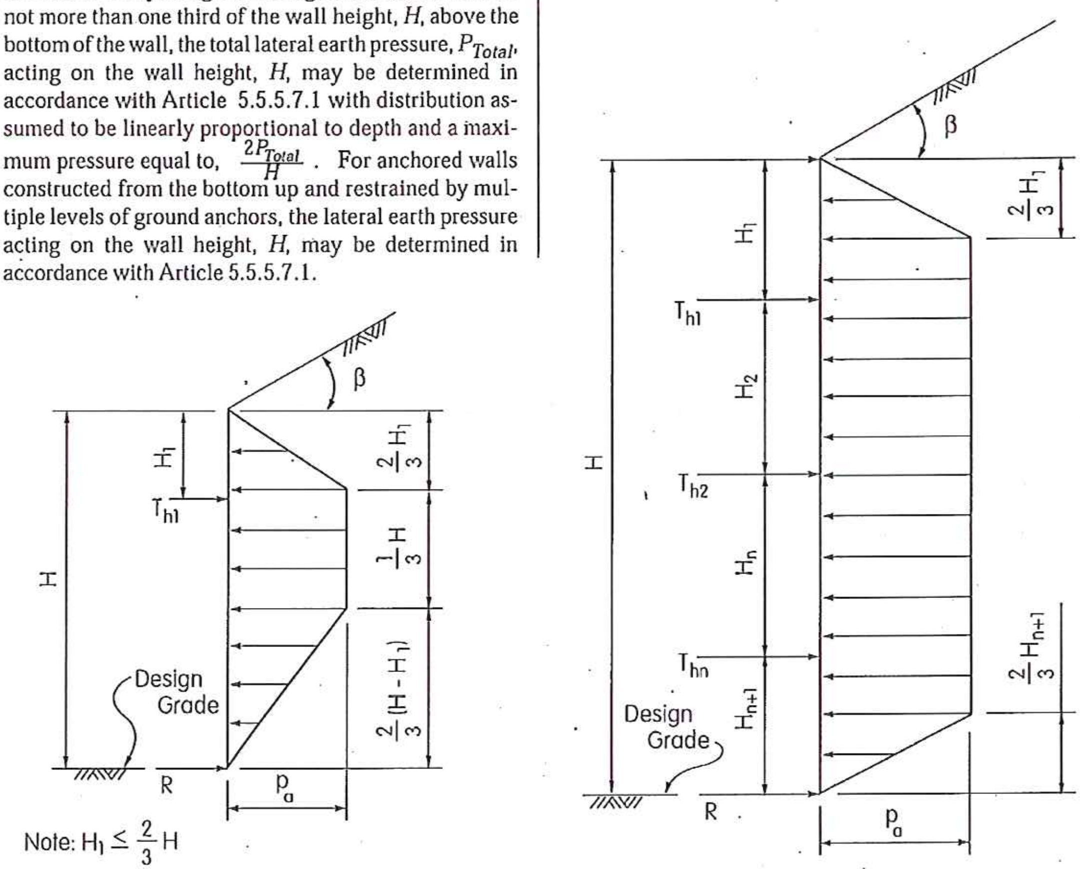
For anchored walls constructed from the top down and restrained by ground anchors (tieback anchors), the lateral earth pressure acting on the wall height, H, may be determined in accordance with Articles 5.5.5.7.1 and 5.5.5.7.2.

For anchored walls constructed from the bottom up and restrained by a single level of ground anchors located not more than one third of the wall height, H, above the bottom of the wall, the total lateral earth pressure, P_{Total}, acting on the wall height, H, may be determined in accordance with Article 5.5.5.7.1 with distribution assumed to be linearly proportional to depth and a maximum pressure equal to, $\frac{2P_{Total}}{H}$. For anchored walls constructed from the bottom up and restrained by multiple levels of ground anchors, the lateral earth pressure acting on the wall height, H, may be determined in accordance with Article 5.5.5.7.1.

In developing the lateral earth pressure for design of an anchored wall, consideration shall be given to wall displacements that may affect adjacent structures and/or underground utilities.

C5.5.5.7

In the development of lateral earth pressures, the method and sequence of wall construction, the rigidity of the wall/anchor system, the physical characteristics and stability of the ground mass to be supported/retained, allowable wall deflections, anchor spacing and prestress and the potential for anchor yield should be considered.



a) Wall with a single level of anchors

b) Wall with multiple levels of anchors

Figure 5.5.5.7.1-1 Lateral Earth Pressure Distributions for Anchored Walls Constructed from the Top Down in Cohesionless Soils

5.5.5.7.1 Cohesionless Soils

The lateral earth pressure distribution for the design of temporary or permanent anchored walls constructed in cohesionless soils may be determined using Figure 5.5.5.7.1-1, for which the maximum ordinate, p_a, of the pressure diagram is determined as follows:

For walls with a single level of anchors :

$$p_a = \frac{P_{Total}}{\frac{2}{3}H} \qquad (5.5.5.7.1\text{-}1)$$

For walls with multiple levels of anchors:

$$p_a = \frac{P_{Total}}{\left(H - \frac{1}{3}H_1 - \frac{1}{3}H_{n+1}\right)} \qquad (5.5.5.7.1\text{-}2)$$

where:

p_a = maximum ordinate of pressure dia gram (KSF)

P_{Total} = total lateral load required to be ap plied to the wall face to provide a factor of safety equal to 1.3 for the retained soil mass when stability is analyzed using an appropriate limiting equilibrium method of analysis. Except that P_{Total}, shall not be less than 1.44 P_a. (KIP)

P_a = active lateral earth pressure resultant acting on the wall height, H, and determined using Coulomb's theory with a wall friction angle, δ, equal to zero. (KIP)

H = wall design height (FT)

H_1 = distance from ground surface at top of wall to uppermost level of anchors. (FT)

H_{n+1} = distance from design grade at bottom of a wall to lowermost level of anchors (FT)

T_{hi} = horizontal component of design force in anchor at level i (KIP/FT)

R = design reaction force at design grade at bottom of wall to be resisted by embedded portion of wall (KIP/FT)

5.5.5.7.2 Cohesive Soils

The lateral earth pressure distribution for cohesive soils is related to the stability number, N_S, which is defined as:

$$N_s = \frac{\gamma_s H}{S_u}$$

where:

γ_s = total unit weight of soil (KCF)

H = wall design height (FT)

S_u = average undrained shear strength of soil (KSF)

5.5.5.7.2a Stiff to Hard

For temporary anchored walls in stiff to hard cohesive soils ($N_s \le 4$), and β=zero, the lateral earth pressure may be determined using Figure 5.5.5.7.1-1, with the maximum ordinate, p_a, of the pressure diagram determined as:

$$p_a = 0.2\,\gamma_s H \text{ to } 0.4\,\gamma_s H \qquad (5.5.5.7.2a-1)$$

where:

p_a = maximum ordinate of pressure diagram (KSF)

γ_s = total unit weight of soil (KCF)

H = wall design height (FT)

For permanent anchored walls in stiff to hard cohesive soils, the lateral earth pressure distributions described in Article 5.5.5.7.1 may be used with, P_{Total}, based on the drained friction angle of the cohesive soil. For permanent walls, the distribution (permanent or temporary) resulting in the maximum total force shall be used for design.

DIST.	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Ala	13	8.3		

REGISTERED ENGINEER - CIVIL 01-23-13

PLANS APPROVAL DATE

Eduardo Ortega No. 41012 Exp. 03-31-13 CIVIL STATE OF CALIFORNIA REGISTERED PROFESSIONAL ENGINEER

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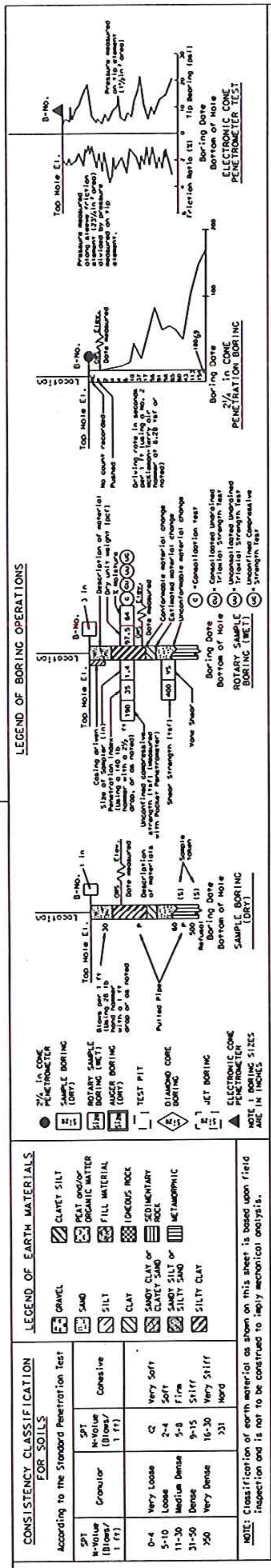

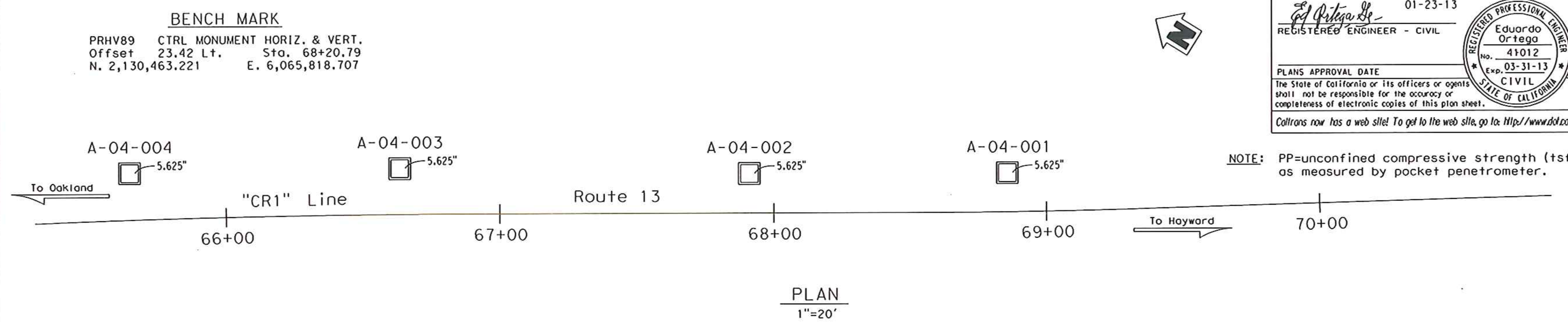

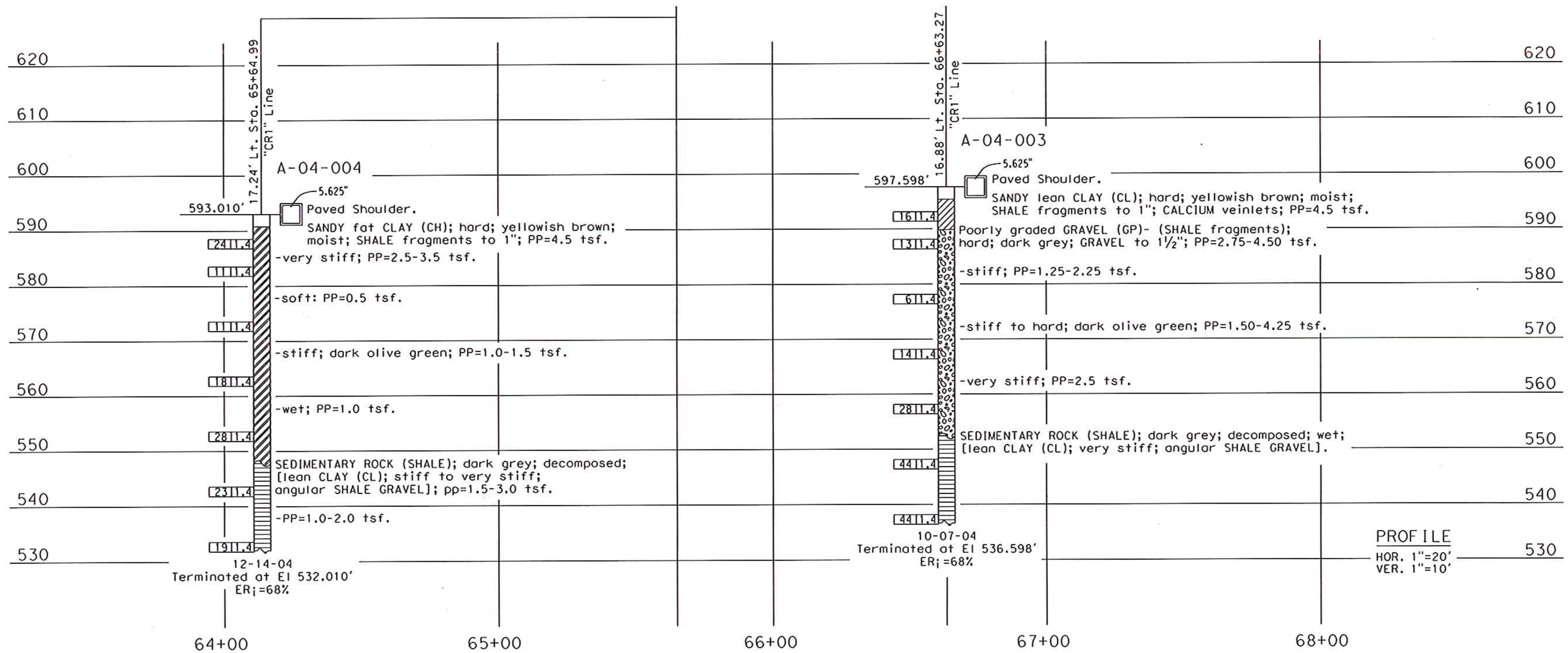

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DRAWN BY: M. REYNOLDS 08/12	E. Ortega				LOG OF TEST BORINGS 1 of 2
CHECKED BY: R. Nashed					

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

UNIT: 3660
EA 04-1SS020

DISREGARD PRINTS BEARING EARLIER REVISION DATES 01-30-13

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04	Ala	13	8.3		

REGISTERED ENGINEER - CIVIL 01-23-13

Eduardo Ortego No. 41012 Exp. 03-31-13 CIVIL STATE OF CALIFORNIA REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

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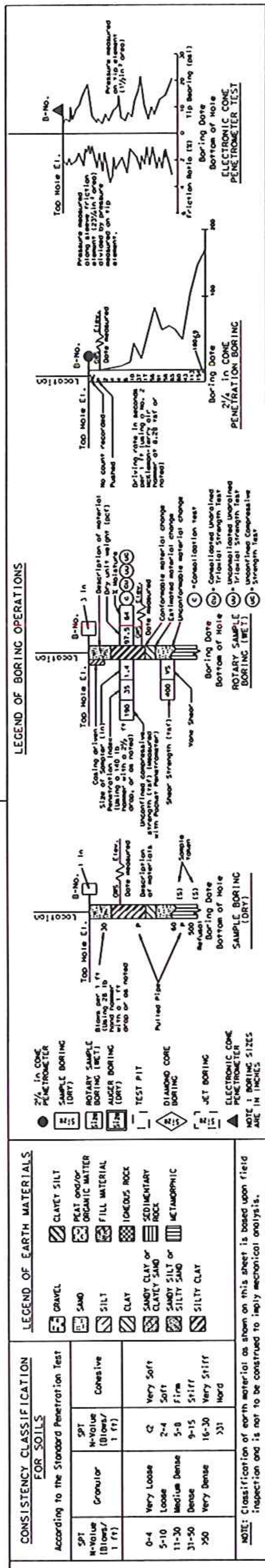
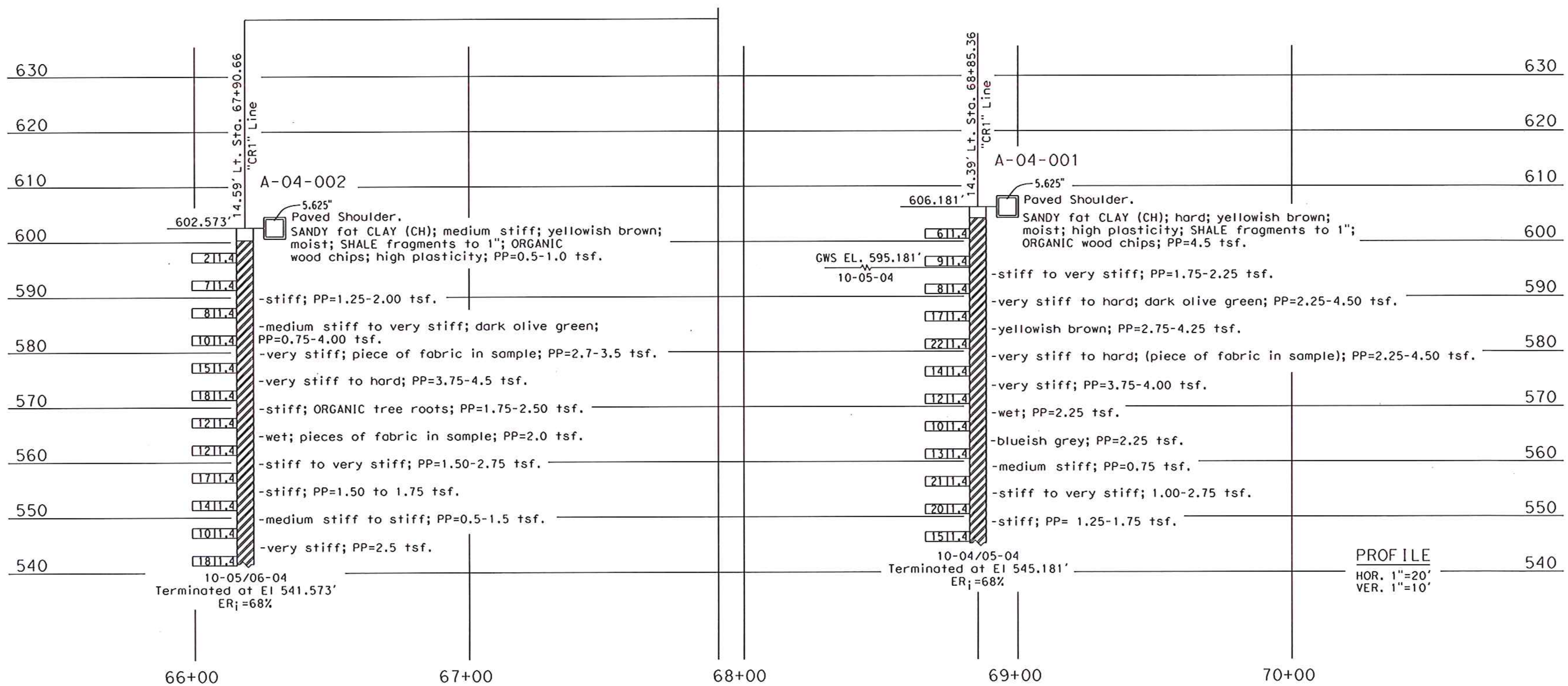
(For Boring Location See Plan, LOTB Sheet 1 of 2)

NOTE: PP=unconfined compressive strength (tsf) as measured by pocket penetrometer.





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DRAWN BY	M. REYNOLDS 08/12	E. Ortego				LOG OF TEST BORINGS 2 of 2
CHECKED BY	R. Nashed					

(Orig 01/02) (Rev 09/12)

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

UNIT: 3660
EA 04-1SS020

DISREGARD PRINTS BEARING EARLIER REVISION DATES

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04	Ala	13	8.3		

Ed Ortega Ge 01-23-13
REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Eduardo Ortega No. 41012 Exp 03-31-13 CIVIL STATE OF CALIFORNIA

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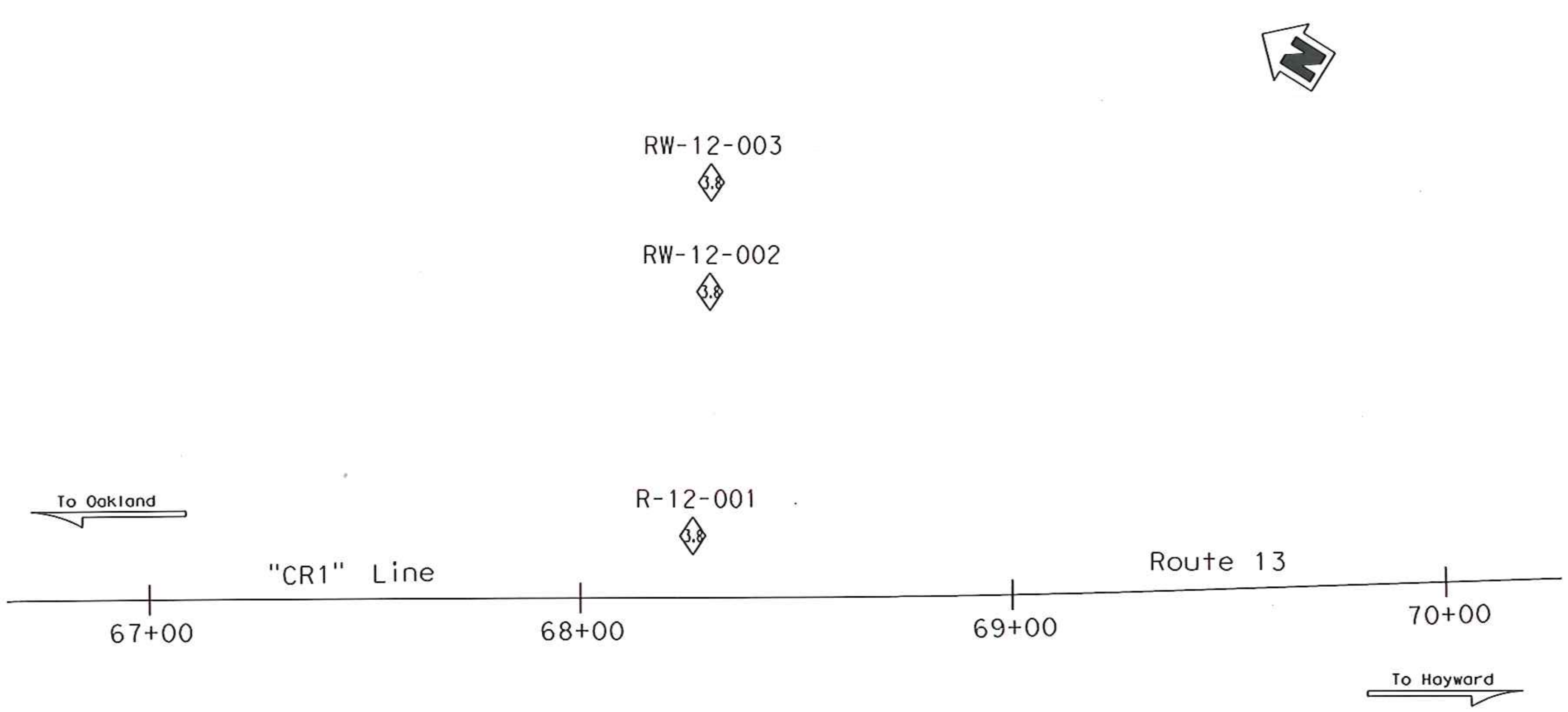
This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

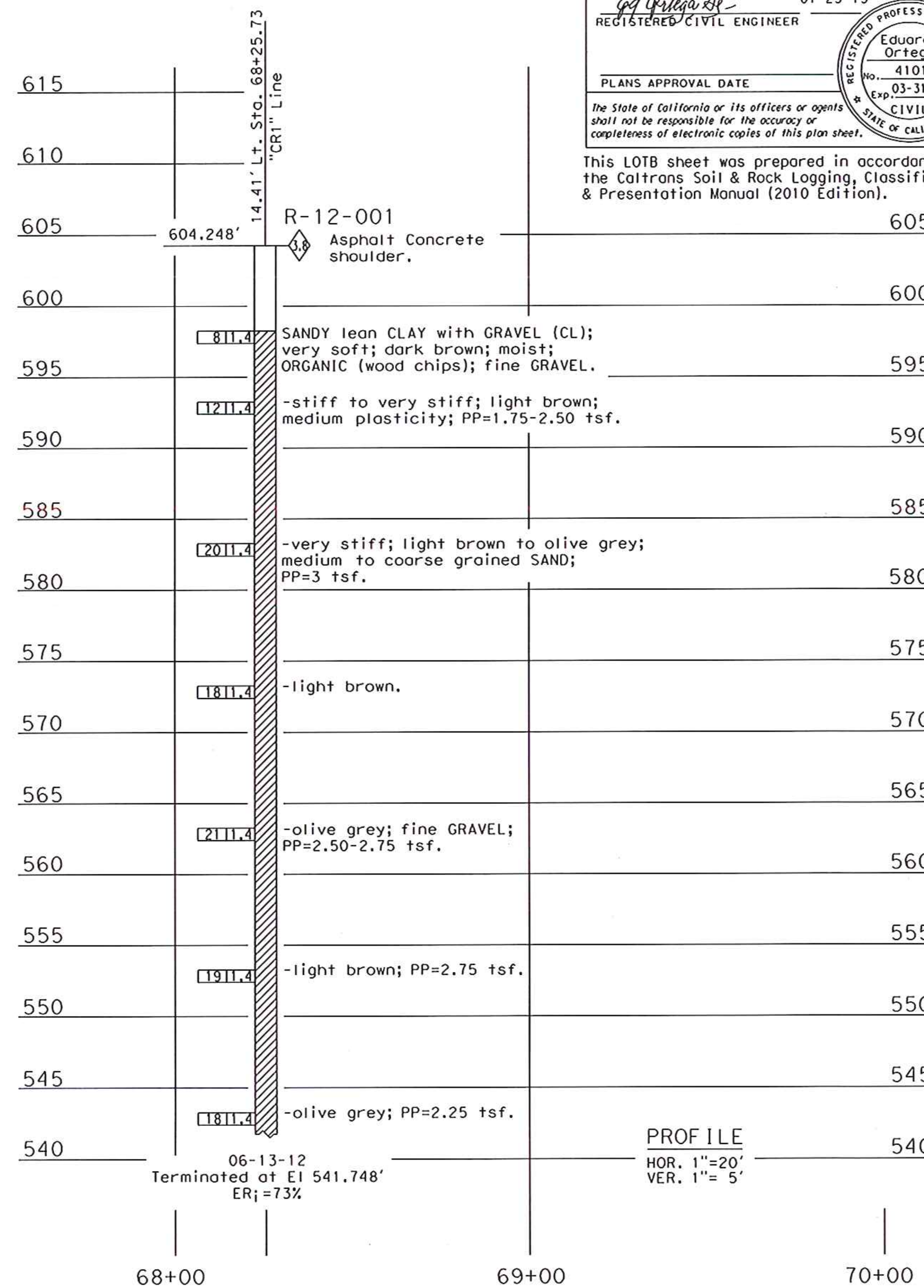
BENCH MARK

PRHV89 CTRL MONUMENT HORIZ. & VERT.
Offset 23.42 Lt. Sta. 68+20.79
N. 2,130,463.221 E. 6,065,818.707



PLAN
1"=20'

NOTE: PP=unconfined compressive strength (tsf) as measured by pocket penetrometer.



PROFILE
HOR. 1"=20'
VER. 1"= 5'

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL DESIGN BRANCH	BRIDGE NO. / POST MILES 8.3	THORNHILL/FERNWOOD RETAINING WALLS 2012 LOTB
FUNCTIONAL SUPERVISOR NAME: M. Momenzadeh	DRAWN BY: M. Reynolds CHECKED BY: R. Nashed	FIELD INVESTIGATION BY: E. Ortega				LOG OF TEST BORINGS 1 OF 2

OGS CIVIL LOG OF TEST BORINGS SHEET — ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 3660
PROJECT NUMBER & PHASE: 04000208680 CONTRACT NO.: 04-1SS020
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REGISTERED CIVIL ENGINEER 01-23-13

PLANS APPROVAL DATE

Eduardo Ortego No. 41012 Exp. 03-31-13 CIVIL — REGISTERED PROFESSIONAL ENGINEER — STATE OF CALIFORNIA

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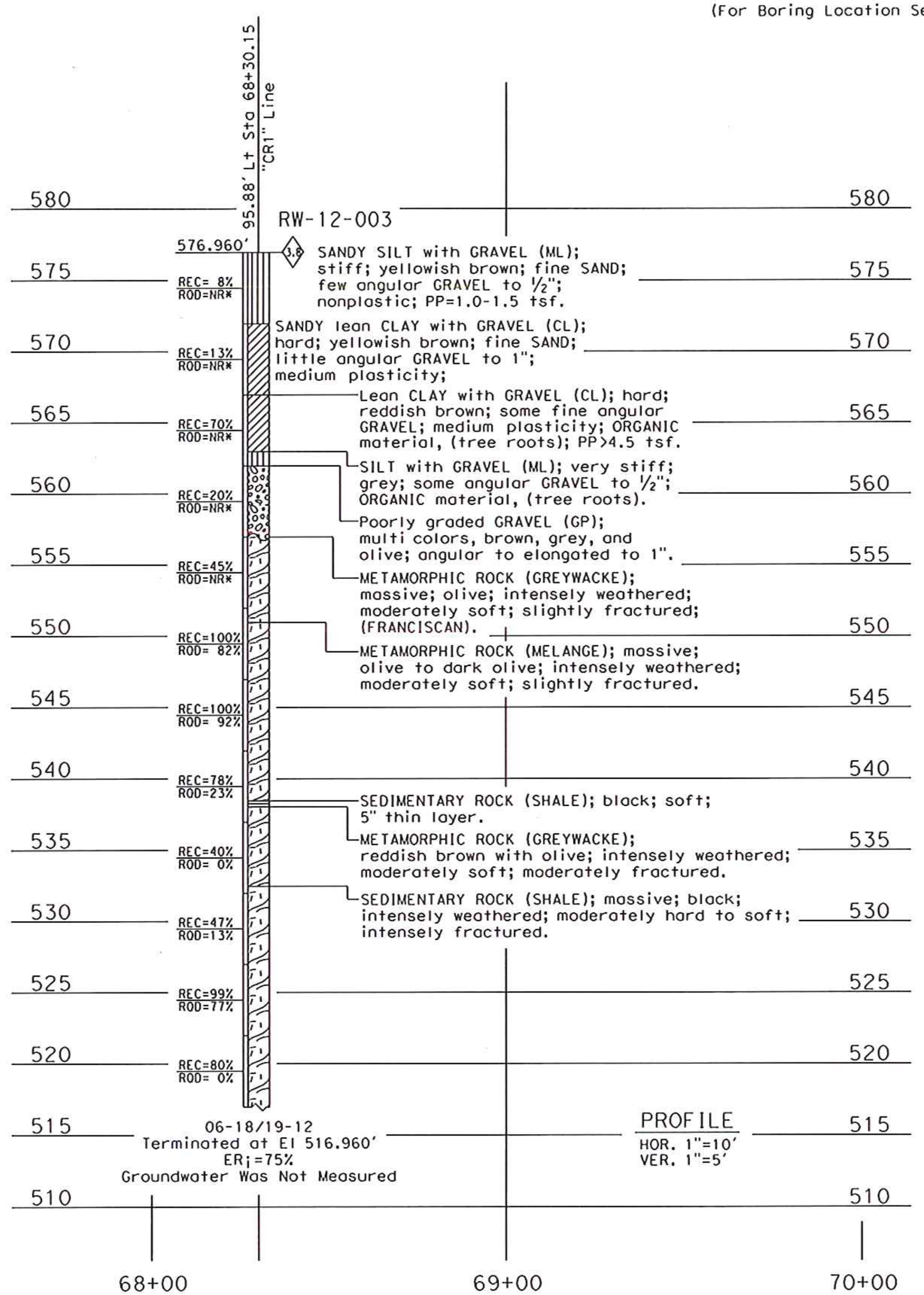
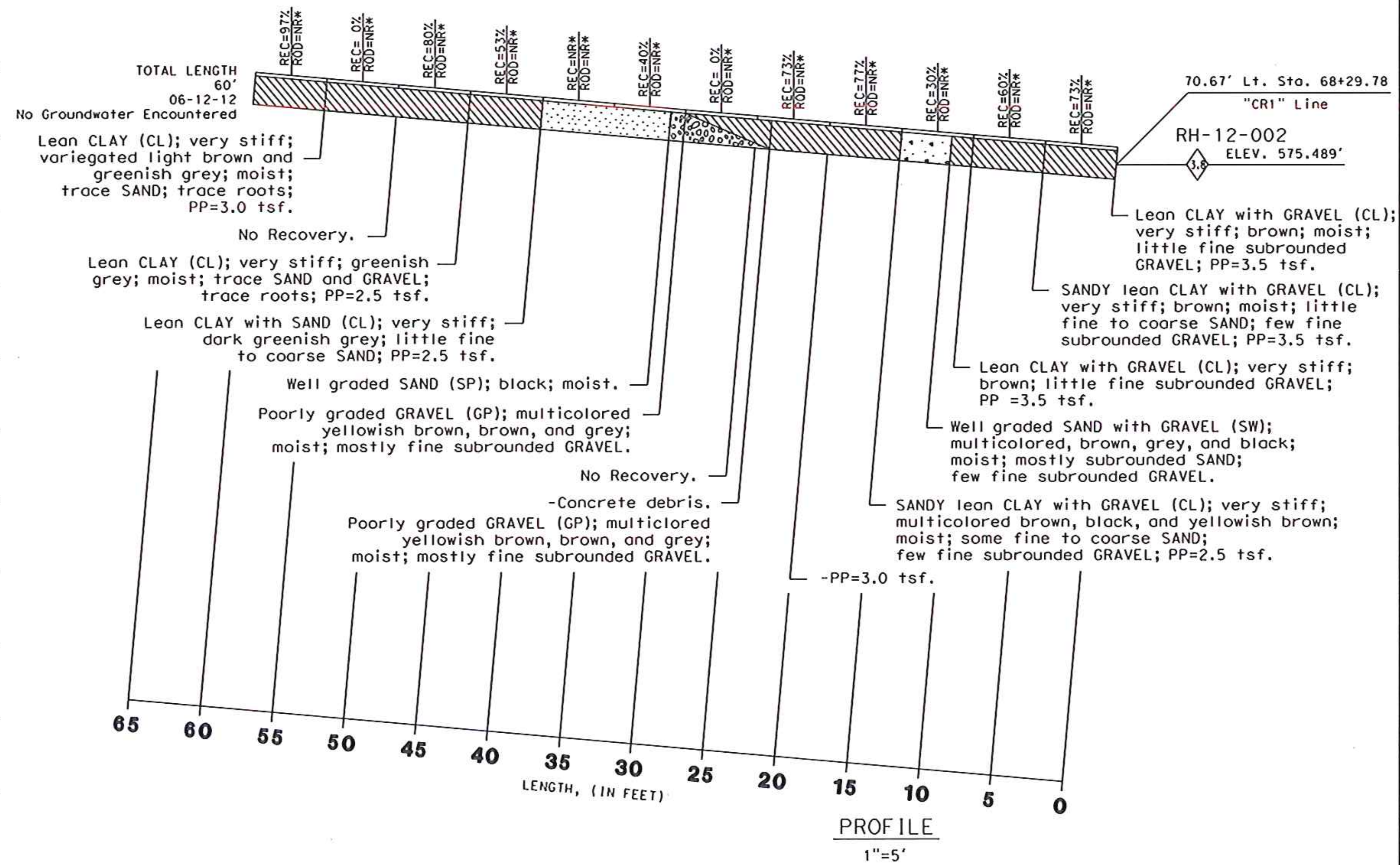
NOTE: 1. PP=unconfined compressive strength (tsf) as measured by pocket penetrometer.

2. *NR: Not Recorded.

(For Boring Location See Plan, LOTB Sheet 1 of 2)





ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL DESIGN BRANCH	BRIDGE NO. / POST MILES 8.3	THORNHILL/FERNWOOD RETAINING WALLS 2012 LOTB — LOG OF TEST BORINGS 2 OF 2
FUNCTIONAL SUPERVISOR	DRAWN BY: M. Reynolds	FIELD INVESTIGATION BY: R. Nashed				
NAME: M. Momenzadeh	CHECKED BY: R. Nashed					

OGS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 3660

PROJECT NUMBER & PHASE: 04000208680 CONTRACT NO.: 04-1SS020

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SLOPE INCLINOMETERS AND PIEZOMETERS INFORMATION

Moraga Ave Repair Project (04-1ss021)

04-ALA-13, PM 8.3

Slope Inclinometer and Piezometer (Well) Destruction

A total of eight slope inclinometers (SI) were installed to monitor the slide. The SI's were installed after drilling a 3" diameter soil boring. The SI depths range from 48 to 58 ft (see attached spreadsheet). Four SI's are installed on the NB on-ramp to Route 13, and four were installed at the bottom of the slope near Fernwood Drive. SI-12B was destroyed by not documented in 2012. SI-12B will be drilled out to a depth of 50 feet and grouted. SI-12B is located 8 feet west of SI-12 (see attached site map).

A total of five piezometers were installed to measure the groundwater elevation. The piezometers were installed in the median of Route 13 (see attached site map). The piezometers were installed after drilling a 3" diameter soil boring. The piezometer depths range from 50 to 60 ft.

The piezometers and slope inclinometers will be destroyed by drilling a hole slightly larger than the original soil boring. The plastic pipe used for the piezometers and SI's will be removed during the drilling operation, and the boring will be grouted in accordance with the Local Enforcement Agency (LEA).

Moraga Ave Slope Repair
EA: 04-1ss021
04-ALA-13 PM 8.3

Piezometer Information

Piezometer	Depth (ft)
P-8	60
P-9	50
P-10	50
P-11	50
P-12	50
Total	260 ft

Slope Inclinometer (SI) Information

Slope Inclinometer	Depth (ft)
SI-7	56
SI-8	56
SI-9B	52
SI-11	48
SI-12B *	Destroyed
SI-12	50
SI-13	50
SI-14	58
Total	370 ft

* SI-12B destroyed in 2012, but not documented.
Backfilled boring will be drilled and grouted in complanice with LEA requirements.
The backfilled boring will be drilled to a depth of 50 ft.

SITE MAP


SI-9B
SI-12
SI-13
SI-11
Fernwood Dr
HD-1
HD-2
HD-3
HD-4
P-12
P-11
P-10
P-8
P-9
SI-8
SI-14
SI-7

Moraga Ave Repair Project (04-1ss021)

04-ALA-13, PM 8.3

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Moraga Ave Slope Repair
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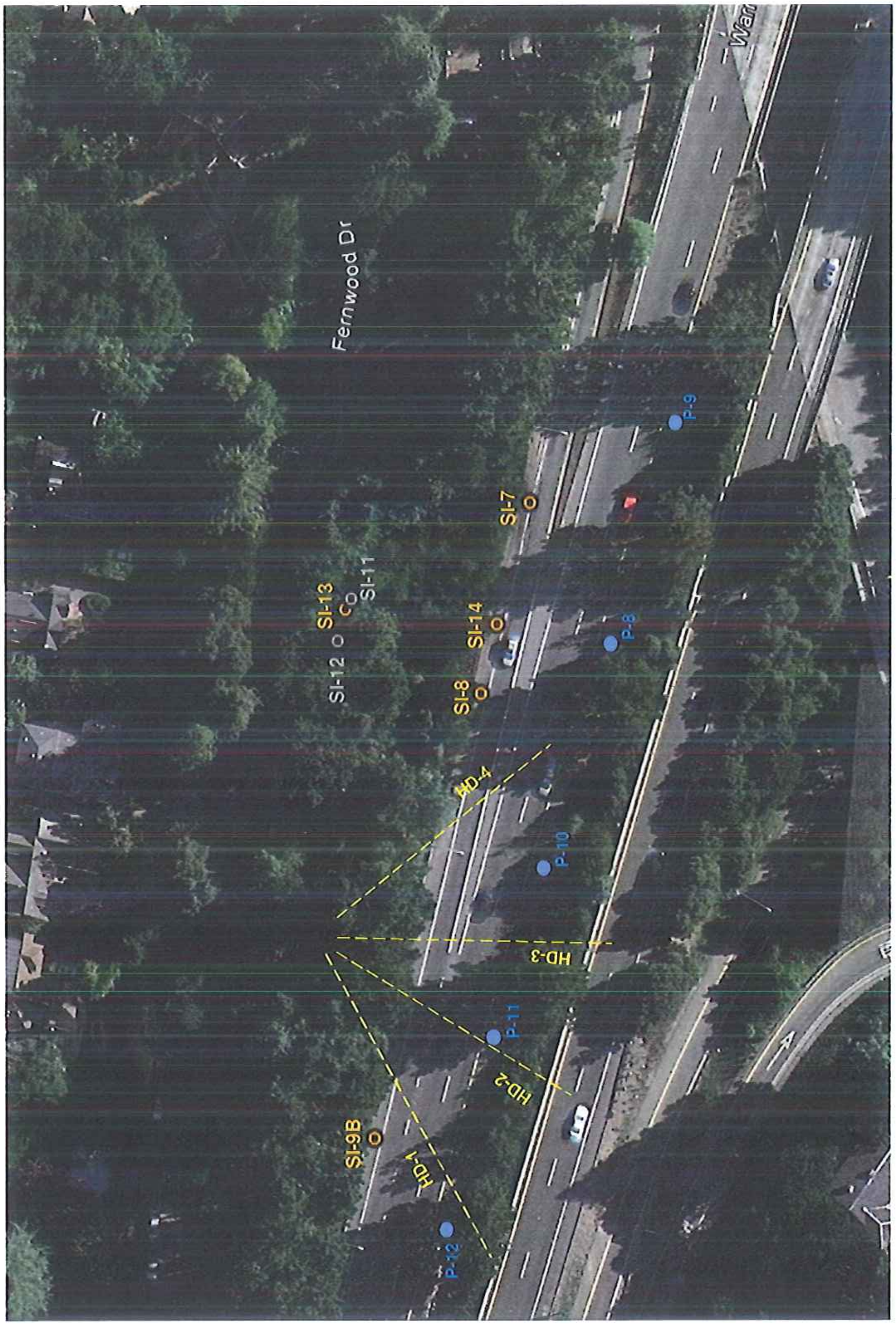
Backfilled boring will be drilled and grouted in complanice with LEA requirements.
The backfilled boring will be drilled to a depth of 50 ft.

SITE MAP


Fernwood Dr
SI-7
P-9
SI-13
SI-11
SI-12
SI-14
SI-8
P-8
HD-4
P-10
HD-3
P-11
HD-2
SI-9B
HD-1
P-12