State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

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Be energy efficient!

To: MR. MIKE KEEVER
Supervising Bridge Engineer
Bridge Design West
Structures Design

Date: June 18, 2013

File: 4-CC-24-PM 5.1/5.6
04 – 3G1601
Efis-0412000004-1
Slope Stressing

Attention: I. Yalan

From: M. ZABOLZADEH/ A. KADDOURA
Associate Materials and Research Engineers
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

HOOSHMAND NIKOUI
Chief, Branch A
Office of Geotechnical Design - West
Geotechnical Services
Division of Engineering Services

Subject: Foundation Report for the Proposed Slope Stressing

1. INTRODUCTION

This memorandum presents our geotechnical recommendations for the design of the proposed slope stressing to address the existing landslides for the above referenced project.

The project is located on eastbound Route 24 near Happy Valley Undercrossing (PM 5.4), 2.3 miles west of the Route 24/680 Interchange, in the Town of Lafayette in Contra Costa County. Route 24 is 4 lane divided highway in each direction. Refer to the attached Figure 1 for vicinity map.

1.1 History

State Route 24 was constructed between 1955 and 1959. In 1967, the route was widened to its current configuration to accommodate the construction and operation of the BART facilities in the center of the route. In 1988 and 1989, in response to landslide movement known as Bin Wall Landslide (For complete history of this landslide see Section 1.3.2 of this report), Caltrans constructed a buried CIDH pile wall near the toe of the fill slope. Also, as part of this project, a broken 27 inches diameter Corrugated Metal Pipe (CMP) culvert that crosses under Route 24 was repaired to stop water leaking into the active landside. The wall and slope have been continuously monitored since that time and the data indicates that landslide movement continued even after the CIDH wall installation. In 2007, District 4 initiated a pavement rehabilitation project. This project did not address the ongoing landslide movement at this site. Shortly after construction completion of pavement rehabilitation project, the landslide movement caused the new pavement to crack and settle by about 2 inches. This caused the head scarp of the landslide (pavement cracks) to reappear on the surface of the new AC overlay. See attached Exhibits A and B for limits and details of the landslide.

In order to temporarily mitigate the above mentioned settlement and to improve ridability, a District Director's Order (DDO) was issued under Contract No. 04-2G6503-ID 0400021073. The DDO work included the use of lightweight polyurethane grout injection to fill and seal the cracks within the slide mass, repair the broken 27-inch CMP (See Section 1.2 of this report) by injecting grout collar around its joints, lift the existing pavement, grinding the AC surfacing and placing new AC overlay. Construction of this DDO was completed in July 2012.

1.2 Existing 27-Inch Culvert

The existing 27-inch CMP culvert crosses under Route 24 and through the active landslide and connecting to a DI at the toe of the fill slope. See Exhibits A and B. During the summer of 2010, the culvert was video inspected by District 4 Maintenance and was determined that the pipe is separated by the ongoing landslide movement. Currently, water collected along the northern shoulder of Route 24 drains into a drop box and is transported to the south under Route 24 through the culvert. From the inspection, it was determined that the pipe is offset both horizontally and vertically by active land sliding at the site. A portion of the surface water transported by this pipe is pouring directly into the back of the active landslide and is likely contributing to the current movement.

This culvert was repaired in 1987 as part of a project to try addressing the landslide movement of the Bin Wall landslide at that time. As mentioned above (Section 1.1), this culvert is repaired again in July 2012.

1.3 Landslide activity and investigations within the project area

This area has a complex landslide history. To study the landslides activities in this area, Caltrans has installed several Slope Inclinometers (SI, see section 5.2 below) in this area. Currently, Office of GDW is monitoring two landslides: a larger unnamed Landslide (See Section 1.3.1 below) and the Bin Wall Landslide (see Section 1.3.2 blow) and that are present within the project area. The Bin Wall Landslide is contained completely within the larger unnamed landslide and is located on the lower eastern limits of this larger landslide complex. The history of both landslides is as follow: See attached Exhibit B.

1.3.1 Unnamed Landslide Complex

The large unnamed landslide complex is poorly understood but is considered to be a very large, slow moving, deep-seated earth flow. See Exhibit A. This landslide crosses under the Bin Wall Landslide at a depth greater than 100 feet below roadway elevation. The presence of this deeper landside is based on the SIs data accumulated in our ongoing geologic study of Route 24 in the vicinity of the Bin Wall landslide site. This landslide is moving at an extremely slow rate and is not expected to change in its nature. Due to the size and depth of this large unnamed landslide complex and its current minimal impact on Route 24, it was concluded that we do not address this slide as part of this project.

1.3.2 Bin Wall Landslide

The Bin Wall landslide is located along eastbound State Route 24, near Happy Valley Undercrossing (PM 5.4), 2.3 miles west of the Route 24/680 Interchange, in the Town of Lafayette in Contra Costa County. See Exhibits A and B. The Bin Wall landslide is a small part of the much larger unnamed complex landslide (described in Section 1.3.1 above) with much faster rate of movement. Our geotechnical investigation revealed that the Bin Wall landslide is in fact riding over (above) the unnamed landslide. The Bin Wall landslide is about 550 ft long, 47 ft wide (at the roadway elevation), and 68 feet deep (indicated by SI) below Route 24 surface elevation. The head scarp of the failure is surfaced on the existing pavement extending to the north into lanes #2, #3, #4. To the south, the failure plane extends down the slope (2H:1V) approximately 240 feet towards Mt. Diablo Boulevard. See attached Exhibit A. The Bin Wall landslide is characterized as a rotational failure likely caused by combination of heavy surface runoff and water leakage from the existing 27 ft broken culvert (described in Section 1.2 above) seeping into the ground, saturating the soil mass, increasing its weight, reducing its shear strength, and developing the failure plain. The Bin Wall landslide is occurring in both the highway fill and in natural rock consisting of sandstone and mudstone. The landslide is called “Bin Wall” because there is an existing 230 ft long (Station 14+60± to 16+90±) Bin Wall at the hinge point of the fill side slope within the limits of the landslide. The Bin Wall landslide is the focus of this report, therefore, from this point on, the word “Landslide” refers only to the Bin Wall landslide.

2. <u>SCOPE OF WORK</u>

The recommendations contained in this report are based on the results from:

- Subsurface explorations performed in March 2012,
- Field mapping,
- Review of existing files,
- Investigations at this site for the existing Bin Wall.
- Preparation of this report.

3. SITE GEOLOGY AND SEISMICITY

3.1 Regional Geology

The site is located in an area of northwest-southeast trending hills and valleys of the Coast Ranges Geomorphic Province between the coastline to the west and the Great Valley physiographic province to the east. This province is characterized by a series of northwesterly trending ridges, faults, and intermountain valleys formed by compression tectonic forces. The site is located about ten miles west of the Great Valley province.

3.2 Site Geology

The project site is underlain by sedimentary and volcanic rocks of Pliocene age, generally undifferentiated and treated slightly differently by different authors. A regional fold is evident, however and rocks that outcrop in the area are distinctly weakly indurated sediments that weather quickly to clayey soils. Refer to the attached Figure 2 (Geology Map).

3.3 Seismicity

The Calaveras is closest to the project site. Refer to Figure 3 (Fault Map) and the attached Final Seismic Design Recommendations" (FSDR) memo dated April 12 2012, by Hossain Salimi, Senior Materials and Research Engineer of OGDW.

3.4 Liquefaction

Liquefaction is a phenomenon in which loose, saturated, fine-grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water; (2) low-density, fine, sandy soils; and, (3) high-intensity ground motion. Saturated, loose and medium dense, cohesionless soils exhibit the liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading.

Based on the "Final Seismic Design Recommendations" (FSDR) memo dated April 12, 2012 by Hossain Salimi, Senior Materials and Research Engineer of OGDW, the potential of liquefaction is minimal.

4. SUBSURFACE INVESTIGATION

The Contra Costa Soil Survey, 1977 lists the entire project area of this report as Altamont-Fontana Series soils (AcF). This soil occurs on 30 to 50% slopes. It consists of 50% Altamont clay and

30% Fontana silty clay loam. The remaining 15% includes Millsholm loam, Capay clay, Lodo clay loam and Rincon clay loam. Altamont soils cover lower part of north-facing slopes. Fontana soils cover ridge tops and south-facing slopes. Soils are ½ to 10 feet thick on slopes and greater than 25 feet thick in accumulations in the valleys. Where soils are bare runoff is rapid and erosion is moderate to high. Their recommended use is for grazing range.

The Office of Geotechnical Design – West, a Division of Engineering Services, investigated the subsurface conditions at the site using an Acker truck mounted drill rig. Three power borings (R-12-001 through R-12-003) were drilled (March 2012) utilizing the rotary wash drilling method with Standard Penetration Test (SPT) sampling within the project limits. R-12-001 and R-12-002 were drilled to the depths of 91.5 and 76.5, respectively in the eastbound shoulder of Route 24 and R-12-003 was drilled to the depth of 51.5 ft in the inside median of eastbound Route 24. Borings R-12-001 and R-12-002 describe the foundation soils/rocks as approximately 35 ft to 40 ft of medium stiff to hard clays with gravel and some sand. This overlies about 5 ft to 10 ft of medium dense clayey sands and gravel. Hard silt lense was encountered in boring R-12-001 between the depths of 65 ft and 80 ft below roadway surface. The remainders of the borings describe the foundation soils/rocks as soft, very intensely to intensely weathered shale. Boring R-12-003 describes the foundation soils at the location of the tiebacks as about 20 of stiff to very stiff clays. This overlies about 10 ft of stiff to very stiff sandy silt with gravel. The remainder of the boring describes the foundation soils/rocks as soft very intensely weathered shale. The unconfined compressive strength of the clayey soils (using a pocket penetrometer) was estimated to range between 1.0 and 4.5 tsf. The SPT blow counts range from 5 to more than 50 (refusal) blows per foot. Refer to the attached Log of Test Boring (LOTB) sheets for details. The LOTB sheets should be included with the contract plans.

Boring R-12-001 was converted to SI/piezometer (SI #26) to continue monitoring the landslide movement and measure the groundwater levels.

Groundwater was measured at borings R-12-001 to be at 50.8 ft (4/18/2012) below roadway surface at the time of drilling. Groundwater was not measured in borings R-12-002 and R-12-003 due to rotary wash drilling method. Refer to the attached LOTB sheets.

4.1 Groundwater

Pump tests from existing Monitoring Wells (MW) along Route 24 at the Lafayette Bin Wall pumped dry immediately and showed slow recharge. Water levels along Route 24 are generally located at 25 to 35 feet below highway elevation.

Groundwater was measured at the existing MW in the median at 31.5’ below roadway elevation at the time of our drilling (March 2012).

5. GEOTECHNICAL TESTING

5.1 Laboratory and In-Situ Testing

Laboratory testing was performed on selected samples of the subsurface materials obtained during our subsurface investigation for corrosion and moisture content. In-situ tests include performing SPT and pocket penetrometer testing on clay soil samples.

5.2 Existing Instruments

There are 25 existing SIs and several MWs within the project limits used to monitor the landslides and GW level at the Lafayette Bin wall. Not all of the SIs are operational. SI #1, #2, #3, #4, #6, #7, and #10 have been covered by recent AC overlay, or destroyed. However, early information is available for them. See attached Exhibit A for approximate location of the existing SIs.

Instrumentation that has been installed within the existing CIDH pile wall located at the toe of the highway fill at the Lafayette Bin Wall site since 1989 has indicated that down slope movement is on the order of about 1 inch in 20 years. Most movement occurred over the first few years after construction of bin wall, but movement has increased in the last 5 years and continues to present date.

6. CORROSION EVALUATION

Corrosion studies are conducted in accordance with the requirements of California Test Method No. 643.

The Department considers the site to be corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

The following table provides our corrosion test summary:

Boring	***SIC Number***	***Sample Depth***	***Resistivity (Ohm-Cm)***	***pH***	***Chloride Content (ppm)***	***Sulfate Content (ppm)***
R-12-002	c634920	30-50’	1037	7.8		

Note: *Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.*

Based on the laboratory test results on the soil samples, the site appears to be non-corrosive.

7. SLOPE STABILITY ANALYSES (BACK-ANALYSES)

Using Slope W 2007 computer software, we back calculated the soil strength parameters along the failure plane of the sliding mass for a factor of safety of slightly below 1.0. This safety factor was used to simulate the creeping (slide is moving at a very slow rate) movement of the slide mass. For the analysis, we used field measurements, SI data for the landslide, and groundwater to simulate the existing ground condition and slide mass movement into Slope W computer software. Our back analyses show that slide soil material has an effective friction angle of 0° and cohesion of 800 psf along the slip-plane for factor of safety of slightly less than 1.0. The graphical outputs generated by the computer program are attached. According to the LOTB, the soil properties below the slide is estimated to have cohesion of 3000 psf and effective friction angle of 34°.

8. FOUNDATION RECOMMENDATIONS

To address the landslide, we considered two structural repair strategy alternatives: Tieback wall and a combination of soil nail walls and slope stressing. However, per our discussion with Structures Design, tieback wall alternative was eliminated because the depth of the failure plane is significant (68' ± deep at worst area). Thus, the combination of soil nail walls and slope stressing alternative is considered to be the most feasible and effective alternative. This strategy will reduce the driving force of the landslide by removing part of the slide mass (using soil nail walls) from the active zone and anchor the rest of the slide mass (moving zone) to the stable ground (non-moving zone) using slope stressing technique. See attached Exhibit C prepared by Office of Structures Design (OSD).

As shown on the attached Exhibit A, the worst part of the slide is between Stations 13+50± and 16+00± (250'±) on both sides of the existing manhole. Based on our slope stability analysis, we recommend constructing three soil nail walls (SN #1, SN #2, and SN #3) and then constructing three slope stressing walls (Wall #1, Wall #2, and Wall #3) over these soil nail walls to stabilize the worst segment of the landslide area. For the remaining parts of the landslide, we recommend a combination of one or two soil nail walls and one or two slope stressing walls. For design heights and lengths of the proposed walls see Exhibit D.

8.1 Combination of Soil Nail Walls and Slope Stressing Walls

8.1.1 Soil Nail Walls

The purpose of soil nail walls is to stabilize the cut slope during construction and to prevent excessive movement and bearing failure of the foundation soils due to tieback anchors load imposed by slope stressing (See Section 8.1.2 below).

A. Design Criteria for Soil Nail Walls

The design for the proposed soil nail walls is performed using Caltrans' Computer Program "SNAILZWIN", Version 5.1. The rock/soil parameters used in this program were selected based on the vertical borings (See LOTB sheets for details) drilled within the proposed wall limits, and field observations.

The following limiting criteria are used in the design of all proposed soil nail retaining walls:

- The minimum factor of safety with seismic loading (pseudo-static): $FOS_{dynamic} = 1.0$; a horizontal pseudo-static coefficient of 0.20 g was used to simulate seismic loading conditions.

- The maximum spacing of the nails (S_v x S_h),

  $S_{v,MAX} = 5$ ft. S_v is the vertical spacing of the nails.

  $S_{h,MAX} = 6.25$ ft. S_h is the horizontal spacing of the nails.

- The inclination angle (θ) of all the nails to the horizontal = 15 degrees

- The average soil/rock design parameters used for design of each soil nail wall (based on the LOTB sheet) were:

Friction Angle (ϕ)	= 28 degrees
Cohesion (c)	= 1000 psf
Unit Weight (γ)	= 125 pcf

- Soil nail profiles lines shall be parallel to the top of the wall except the bottom most line, which shall be parallel to the bottom of the wall.

  - ❖ Minimum and maximum vertical distances from the bottom of the wall to the bottom level of the soil nail assembly (SB) shall be 1.2 ft and 3 ft, respectively.

  - ❖ Soil nails shall be of ASTM Designation: A615, Grade 75, fs= 75,000 psi and #8 bars for all Soil Nail Walls.

  - ❖ Pullout resistance between grout and drilled hole = 1.6 kips per linear foot of bonded length.

  - ❖ Punching shear capacity = 45 kips.

- ❖ The vertical distance between the bottom of the wall and the finished grade of the proposed bench = 1.5 ft.

- ❖ Vertical distance between top of wall (cut line as shown on the plans) and the top most row of soil nails ST = 1.9 ft.

- ❖ Minimum spacing, both horizontal and vertical, of soil nail assembly = 1.5 ft.

- ❖ Minimum and maximum distances between the beginning/end of wall and the first/last soil nail = 1.5 ft and 6.25 ft, respectively.

- ❖ The designed lengths (embedment depth) of the soil nails will be shown on the proposed Soil Nail Walls Plans when finalized.

B. Field Testing

Field verification of the design pullout resistance values used in the design ensures that the nail design loads can be carried without excessive movements and with an acceptable factor of safety for the service life of the wall. Verification testing and proof testing shall be conducted in order to verify the design pullout resistance and to ensure consistency of the quality of drilling, installation and grouting technique.

Verification testing and stability testing for each "wall zone" shall be conducted prior to the installation of production soil nails in accordance to the special provisions at locations recommended by the Engineer. It is recommended that locations for these tests be shown in the Contractor's working drawing submittal for approval. The wall zones shall be defined as follows:

8.1.2 Slope Stressing Wall

Slope stressing is a tieback system without steel soldier piles and lagging. An individual or continuous concrete waler is constructed over the slope and one or two layers of post tensioned tendons will be used to tie the landslide moving zone to the stable ground below the failure plane.

We recommend that the entire surface of the cut slope including the bearing area of the slope stressing concrete walers be reinforced with soil nails (See Section 8.1.1) and shotcrete surface. We recommend that a continuous waler be used at each of the proposed slope stressing levels in order to distribute the load and deformation more uniformly.

To determine the required anchors loads, we performed slope stability analyses using the site geometry, field measurements; actual slip-plane determined by the existing SIs, back calculated soil/rock parameters, and pour water pressure condition. Below are summary of the minimum required design anchor loads to stabilize the landslide.

Table 1 Tieback Anchor Loadings

TYPE OF LOADING	SF	Tieback Anchor loads
Static	>1.3	T1 = 20 Kips/ft @ 15° angle T2 = 20 Kips/ft @ 15° angle T3 = 20 Kips/ft @ 15° angle T4 = 20 Kips/ft @ 15° angle T5 = 20 Kips/ft @ 15° angle
Seismic	> 1.0	T1 = 20 Kips/ft @ 15° angle T2 = 25 Kips/ft @ 15° angle T3 = 25 Kips/ft @ 15° angle T4 = 25 Kips/ft @ 15° angle T5 = 25 Kips/ft @ 15° angle

The results of the stability analysis are attached

Design Criteria for Slope Stressing Walls

- Use 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 the proposed wall just below the road (first slope stressing wall), include an additional rectangular pressure diagram equivalent to 2 ft of fill from top of the wall to a depth equal to the wall height.

- The walls shall be capable of resisting an additional seismic uniform earth pressure estimated to be equal to 20 H applied 0.6H above the base.

- 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 roadway elevation.

- The proposed first row of tieback anchors should be installed at least 6 ft below the roadway elevation and they should be installed at an angle of 15-25 degrees below the horizontal.

- Below first row vertical distances of tieback rows should be at least 6.0 ft, and they should be installed at an angle of 15-25 degrees below the horizontal.

- The unbonded length of the tieback anchors should be 60 ft and 70 ft as shown on Exhibit D.

- The bonded length of the tieback anchors should be left up to the contractor. The contractor is responsible for providing tieback anchors that satisfy the contract provisions and specifications.

- Tie back horizontal spacing should be limited to not more than 6.5 ft.

9. INSTRUMENTATION

As mentioned in Section 4, SI #26 was installed to replace the sheared SI #1 to monitor the slide movement and GW. Because the existing SIs #26 (SI #1 replacement), #6, and #14 are operational, no additional SI is needed.

10. CONSTRUCTION CONSIDERATIONS

The following construction considerations and requirements should be included in the design and construction specifications for the proposed tieback wall and mitigation measures.

- The Contractor may encounter difficulties during drilling for the subhorizontal ground anchors. This is due to the presence of groundwater and caving soils. Thus, using of casing may be required.

- The anchors for the proposed top two slope stressing walls must be installed before excavating for the third proposed slope stressing wall.

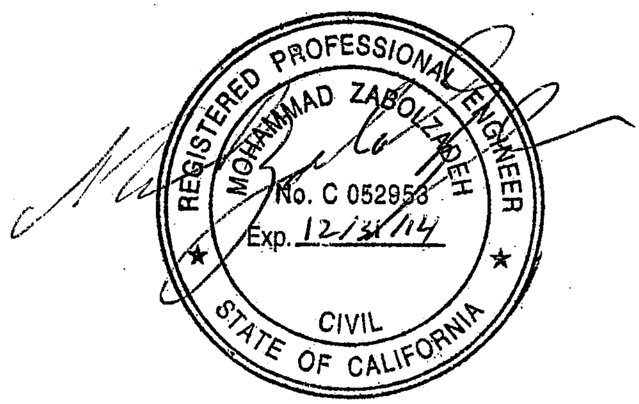
* * * * * *

If you have any questions or need additional information, please call us at (510) 286-4831/4676 or Hooshmand Nikoui, Branch Chief at (510) 286-4811.

Attachments

c: TPokrywka, HNikoui, MZabolzadeh, AKaddoura - (GS west)
SRajendra (GS Support- Office Chief), Structure Construction RE pending File, John Stayton (DES OE), Rubin Woo (District ME), HAlamguer (District 06 PM), SShaath (District 06 PE)

Kaddoura/Zabolzadeh/mm/My Documents/CC-24-PM 5.4 3G1600 Slope Stressing FR.docx



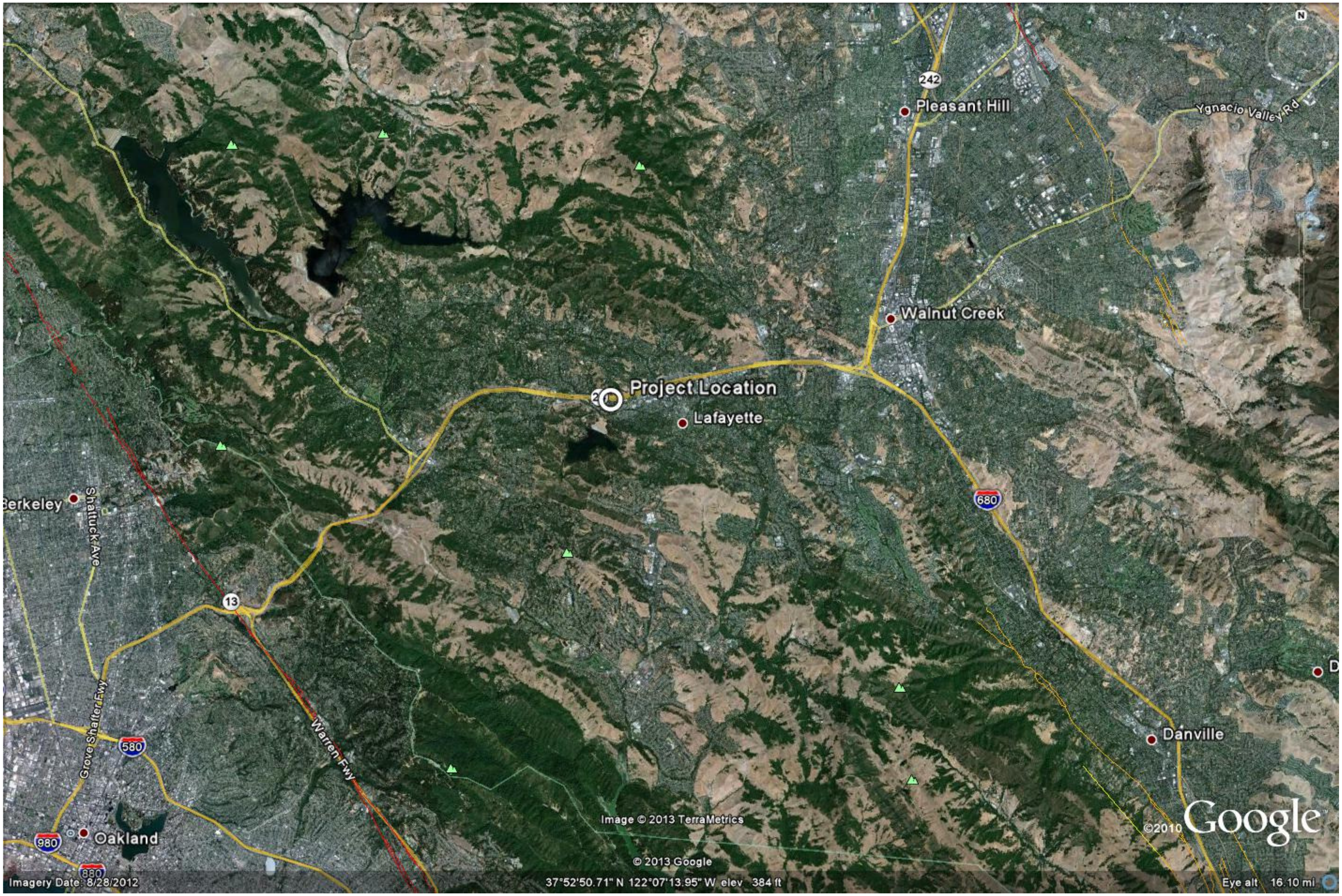


Image courtesy of Google Earth

No Scale

Figure 1 - Vicinity Map	
CC-24	**PM 5.1/5.6**
0412000004-1	**May 2013**

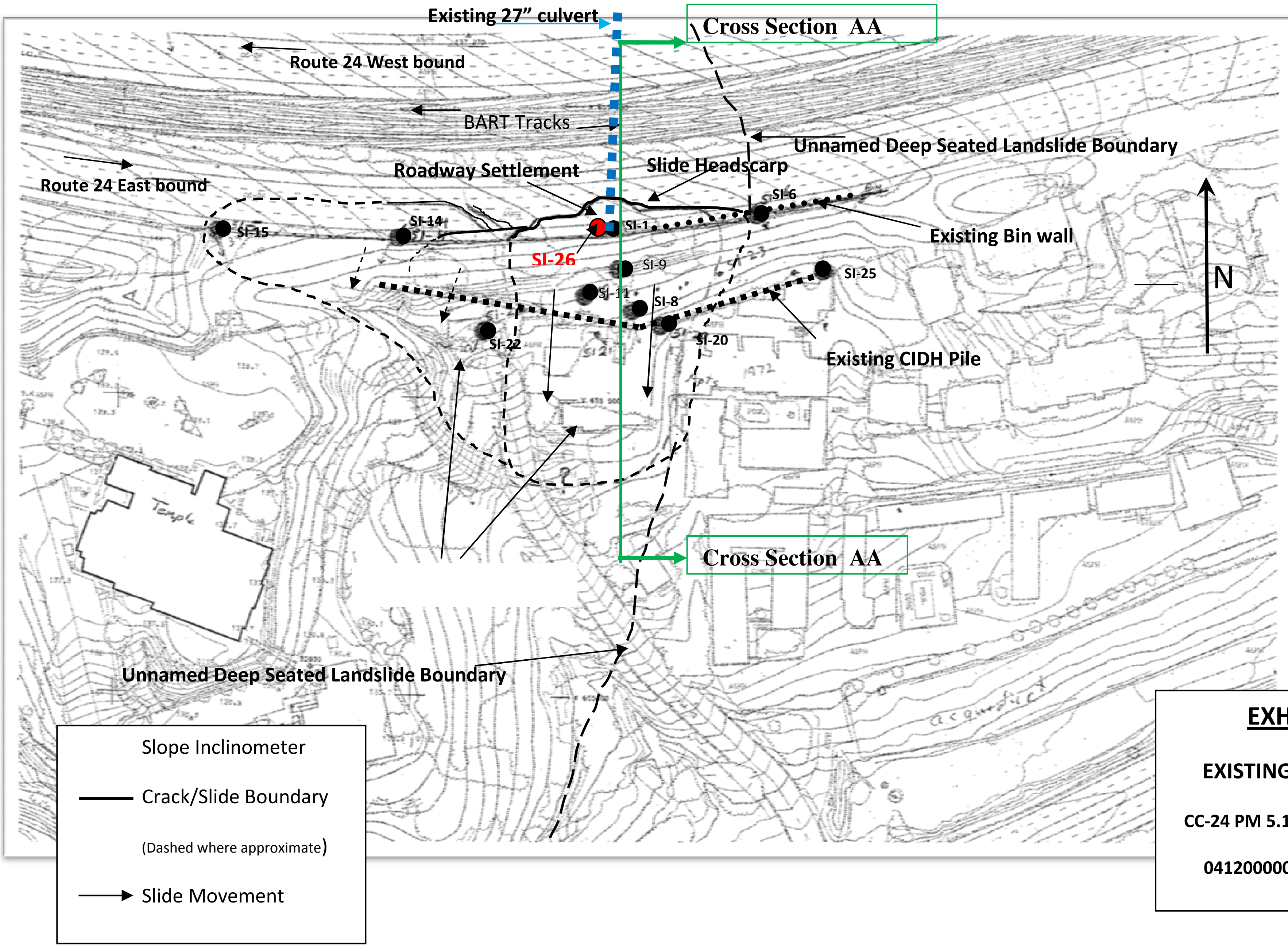


EXHIBIT A

EXISTING LANDSLIDES

CC-24 PM 5.1/5.6 **04-3G1601**

04120000041 **May 2013**

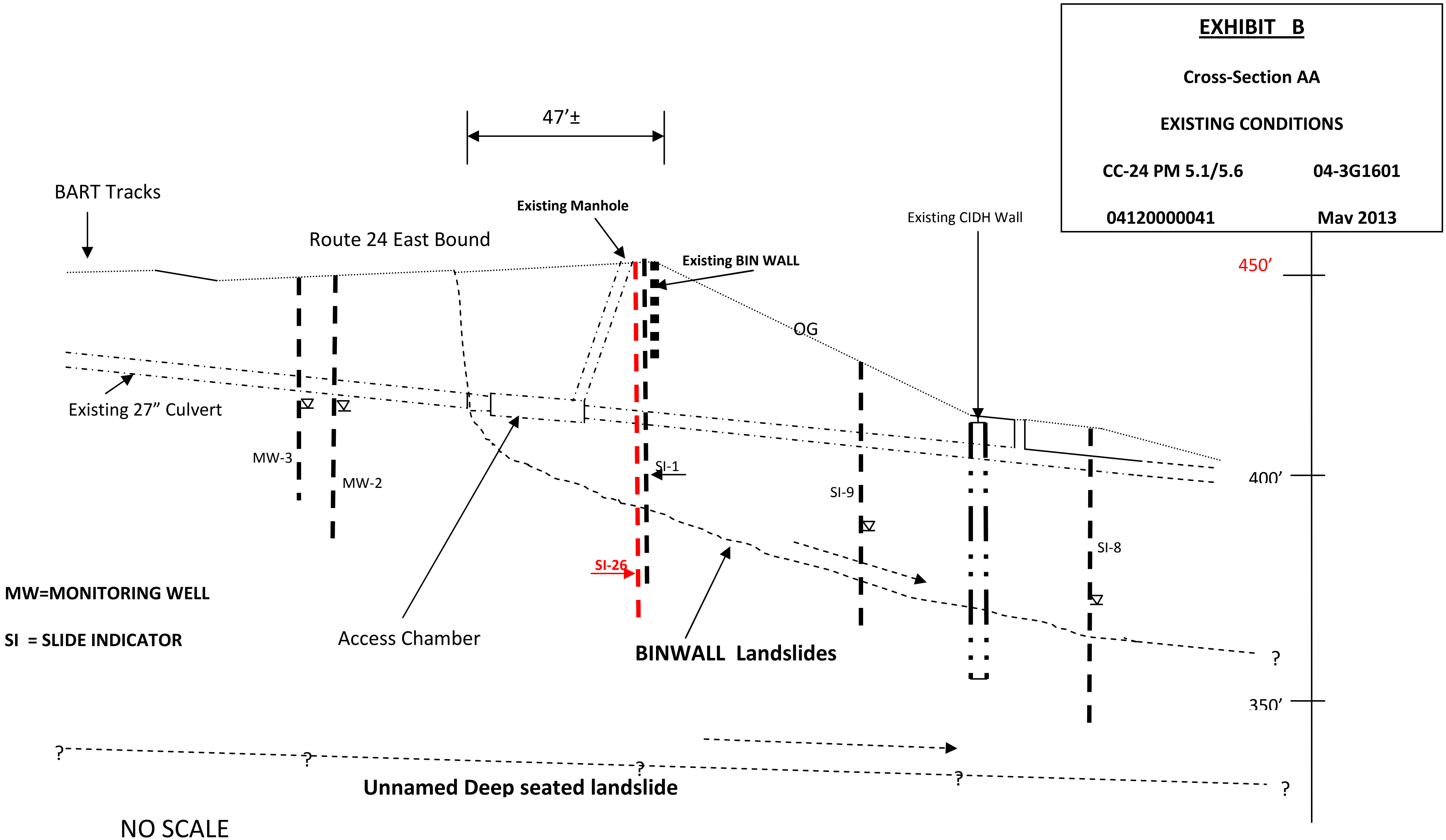
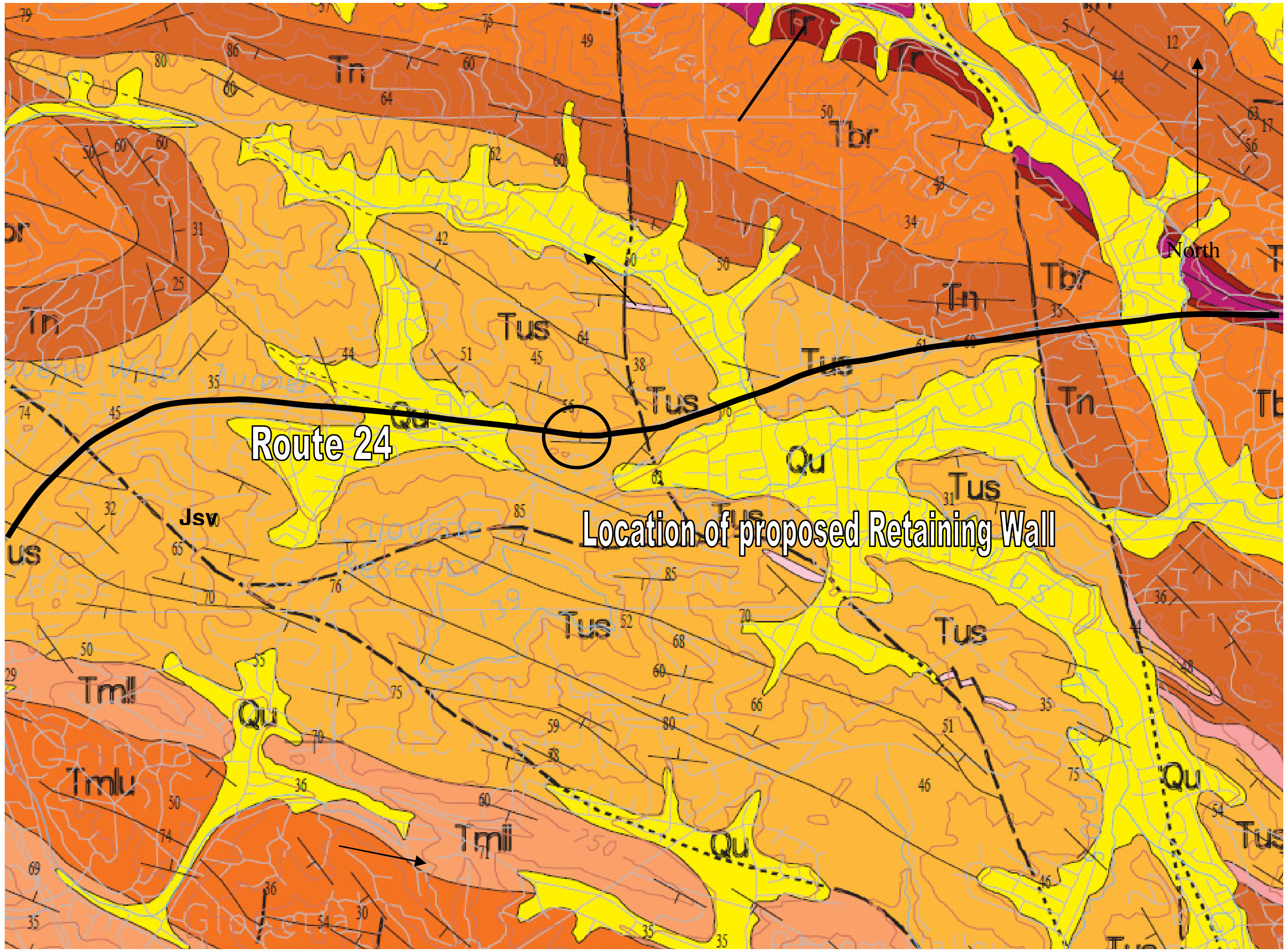


EXHIBIT B
Cross-Section AA
EXISTING CONDITIONS
CC-24 PM 5.1/5.6
04-3G1601
04120000041
May 2013
47'±
BART Tracks
Route 24 East Bound
Existing Manhole
Existing BIN WALL
Existing CIDH Wall
OG
450'
400'
350'
Existing 27" Culvert
MW-3
MW-2
SI-1
SI-26
SI-9
SI-8
MW=MONITORING WELL
SI = SLIDE INDICATOR
Access Chamber
BINWALL Landslides
?
Unnamed Deep seated landslide
NO SCALE



Map Legend

- Qu – Undivided Quaternary deposits
- Qls – Landslide deposits
- Qmz – Montezuma Formation – sand, clay and gravel
- Tmlu – Mulholland Formation – upper member
- Tmll – Mulholland Formation – lower member
- Tus – Unnamed sedimentary and volcanic rocks
- Tul – Limestone member
- Tub – Basalt member
- Tpt – Pinole tuff
- Tlt – Lafayette tuff
- Tn – Neroly Formation – blue sandstone
- Tc – Cierbo Formation – sandstone
- Tbr – Briones Formation, undivided in the southern part divided locally into:
- Tbg – Briones Formation, G member – sandstone and shell breccia
- Tbf – Briones Formation, F member – sandstone and shale
- Tbe – Briones Formation, E member – sandstone and shell breccia
- Tbd – Briones Formation, D member – massive sandstone

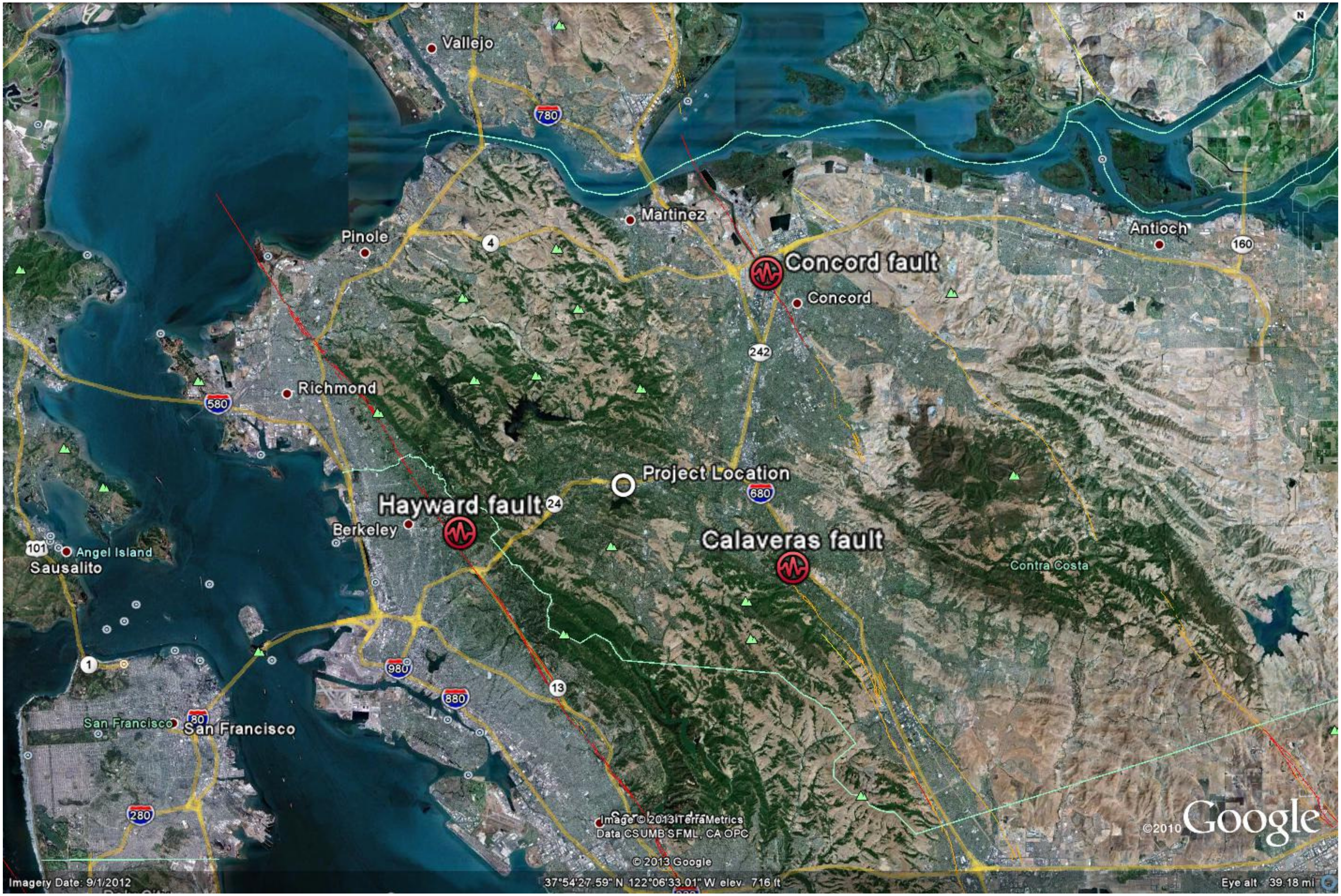
- Contact
- Contact, approximately located
- Contact, inferred
- Contact, uncertain
- Contact, concealed
- Fault
- Fault, approximately located
- Fault, inferred
- Fault, uncertain
- Fault, concealed
- Fault, concealed and uncertain
- Thrust or reverse fault
- Thrust or reverse fault, approximately located
- Thrust or reverse fault, inferred
- Thrust or reverse fault, uncertain
- Thrust or reverse fault, concealed
- Thrust or reverse fault, concealed and uncertain
- Fold axis
- Fold axis, inferred
- Fold axis, concealed
- Strike and dip of bedding
- Strike and dip of overturned bedding
- Strike of vertical bedding

Source: Graymer, R.W., Jones, D.L., and Brabb, E.E., 1994, Preliminary Geologic Map Emphasizing Bedrock Formations in Contra Costa County, California, USGS Open File Report 94-622

Not to scale

Figure 2 - Geology Map

04-CC-24	PM 5.1/5.6
0412000004-1	May 2013

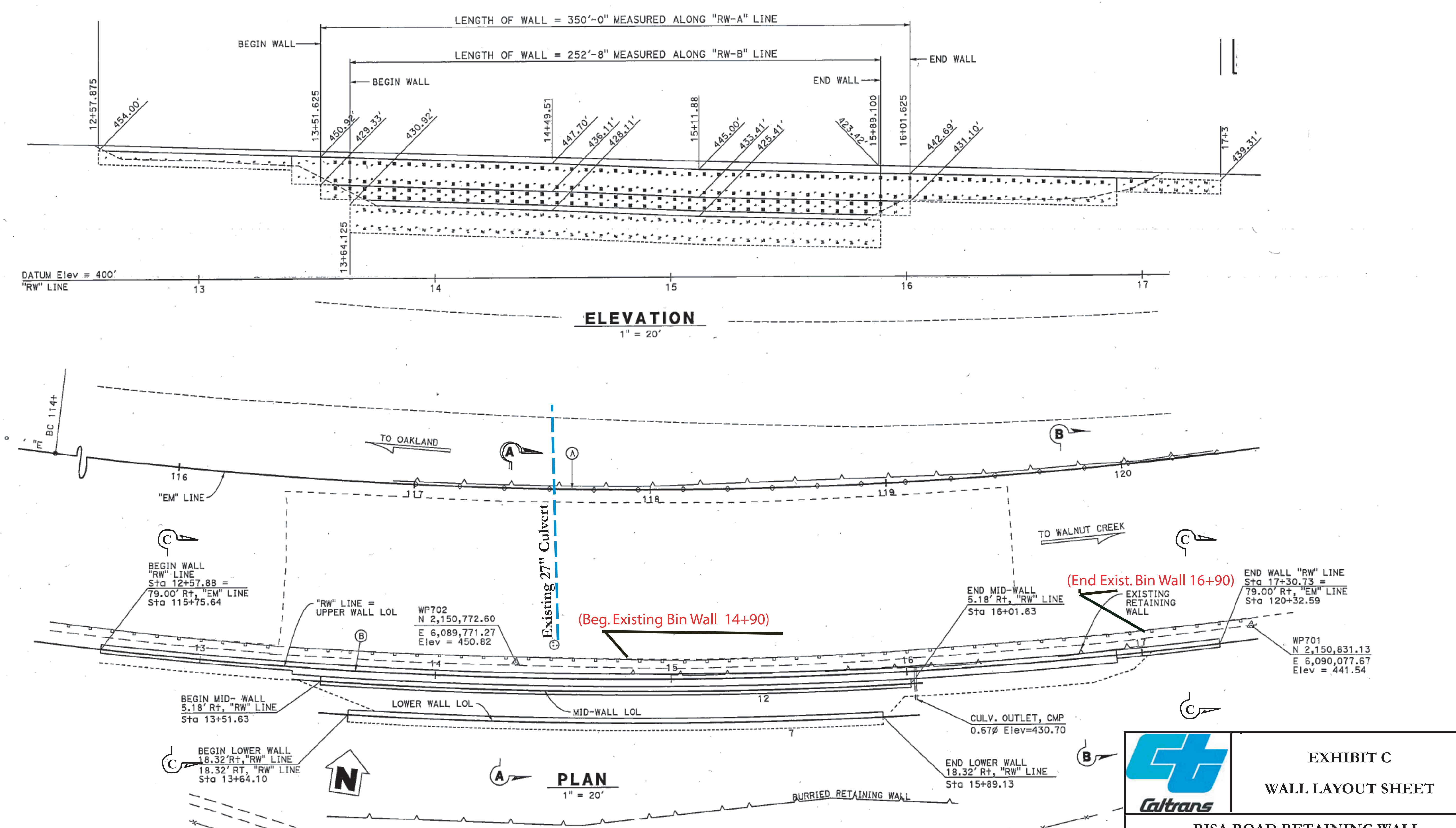


U.S. Geological Survey and California Geological Survey, 2006, Quaternary fault and fold database for the United States, 12/01/2008, from USGS web site: http//earthquakes.usgs.gov/regional/qfaults/
Base map from Google Earth 2012

Caltrans

Figure 3 - Fault Map

04-CC-24	**PM 5.1/5.6**
0412000004-1	**May 2013**



Caltrans

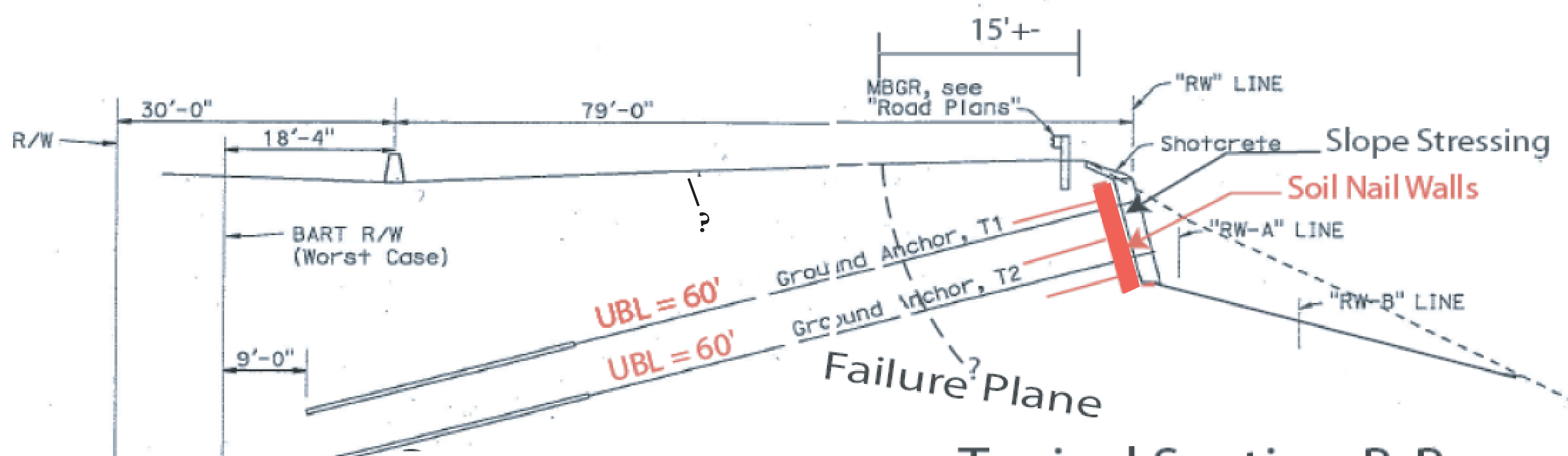
EXHIBIT C

WALL LAYOUT SHEET

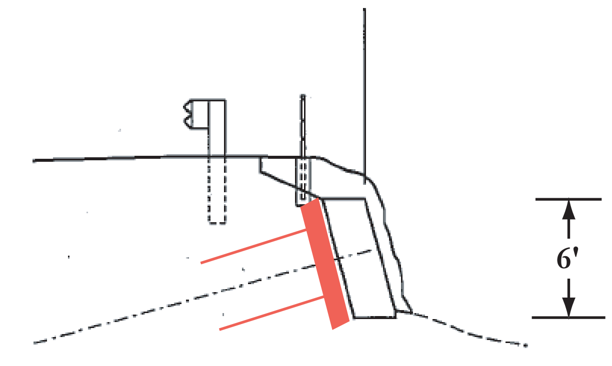
RISA ROAD RETAINING WALL
HIGHWAY 24 P.M. 5.1-5.6
CONTRA COSTA COUNTY, CALIFORNIA

PROJECT NO. 04120000041	MAY 2013	04-3G1601

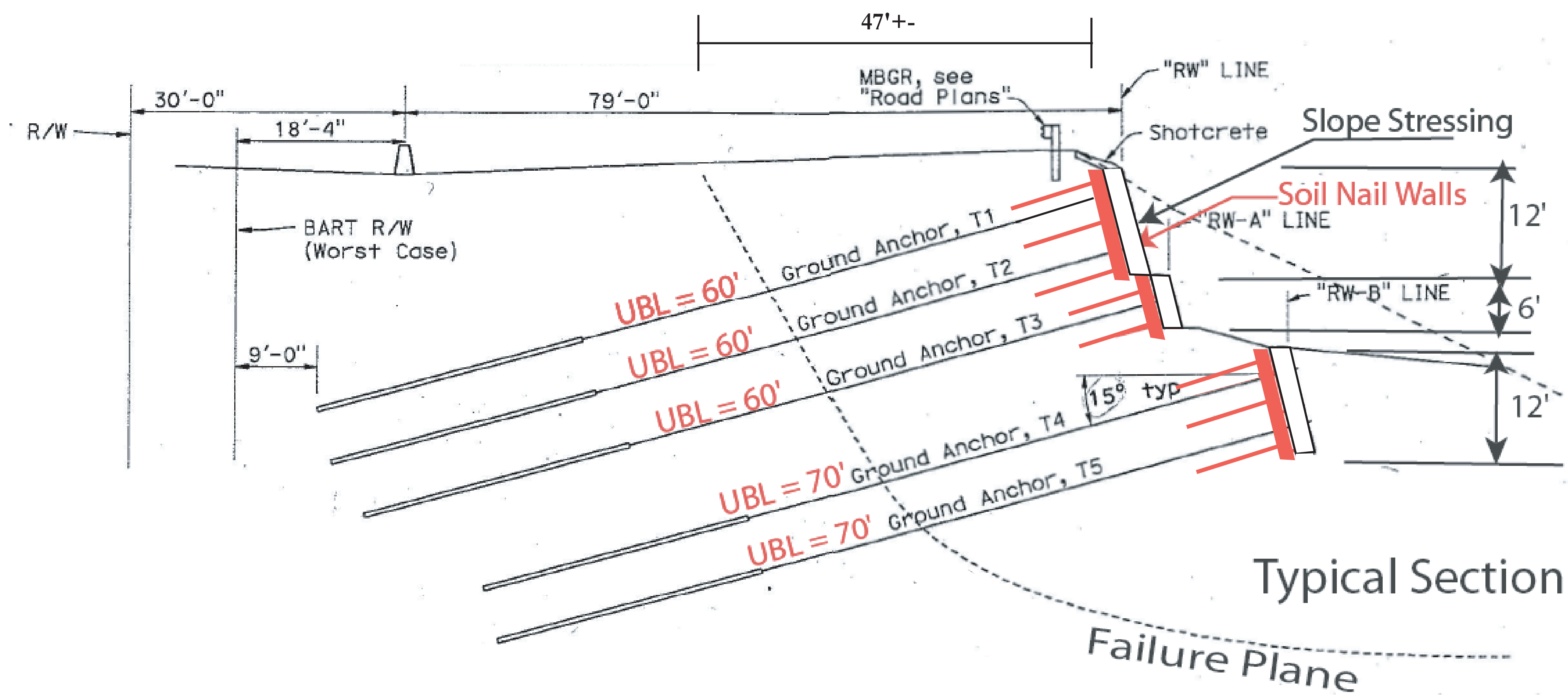
UBL = Unbonded Length



Typical Section B-B



Typical Section C-C



Typical Section A-A

Caltrans	EXHIBIT D TYPICAL CROSS-SECTIONS
RISA ROAD RETAINING WALL HIGHWAY 24 P.M. 5.1-5.6 CONTRA COSTA COUNTY, CALIFORNIA	

PROJECT NO. 04120000041	MAY 2013	04-3G1601

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC	24	5.1/5.6		

Ali Kaddoura 05-24-13
REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
Ali K. Kaddoura
No. 55710
Exp. 12-31-14
CIVIL
STATE OF CALIFORNIA

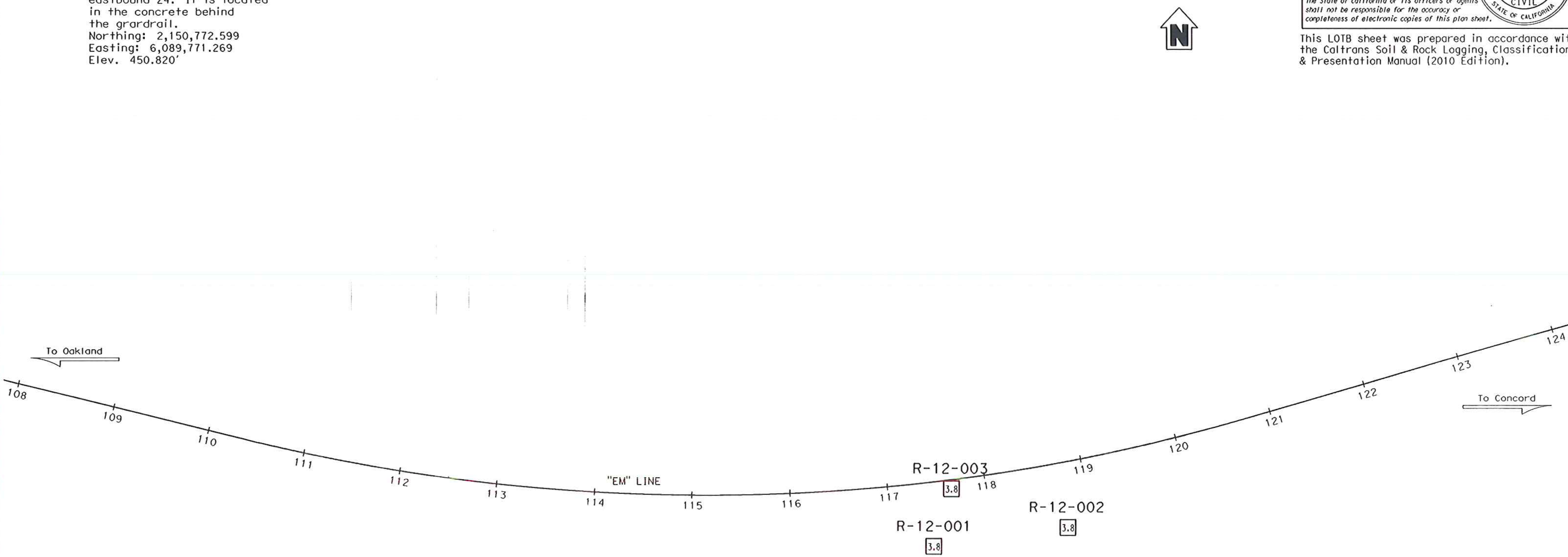
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

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

BENCH MARK

WP 702 is a PK nail + shiner aproximately 325' west of the east end of the guardrail on eastbound 24. It is located in the concrete behind the grardrail.
Northing: 2,150,772.599
Easting: 6,089,771.269
Elev. 450.820'



PLAN
1"=50'

ENGINEERING SERVICES			GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL DESIGN BRANCH	BRIDGE NO. 33E0226	RISA ROAD RETAINING WALL
FUNCTIONAL SUPERVISOR	DRAWN BY: M. Reynolds	08/12	FIELD INVESTIGATION BY: M. Gaffney, A. Kaddoura			POST MILES 5.1/5.6	LOG OF TEST BORINGS 1 of 2
NAME: H. Nikoui	CHECKED BY: R. Nashed						

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 3660
PROJECT NUMBER & PHASE: 04120000041 CONTRACT NO.: 04-3G1601

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES | SHEET | OF

FILE => 43G160qa01.dgn

USERNAME => s110822 DATE PLOTTED => 24-MAY-2013 TIME PLOTTED => 14:17

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	CC	24	5.1/5.6		

Ali Kaddoura 05-24-13
REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
Ali K. Kaddoura
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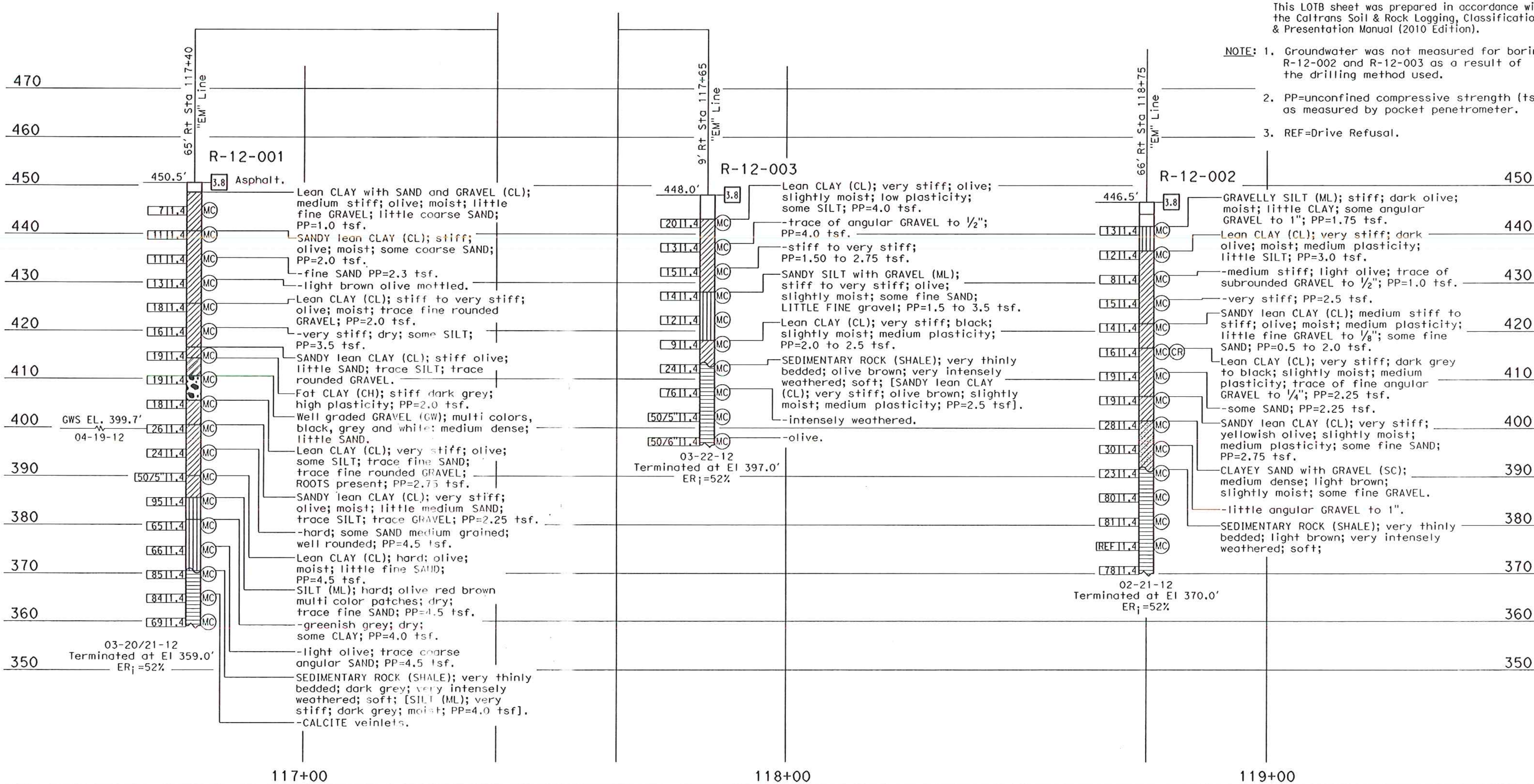
NOTE:
1. Groundwater was not measured for borings R-12-002 and R-12-003 as a result of the drilling method used.
2. PP=unconfined compressive strength (tsf) as measured by pocket penetrometer.
3. REF=Drive Refusal.



ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL DESIGN BRANCH	BRIDGE NO. 33E0226 POST MILES 5.1/5.6	RISA ROAD RETAINING WALL LOG OF TEST BORINGS 2 of 2
FUNCTIONAL SUPERVISOR NAME: H. Nikoui	DRAWN BY: M. Reynolds 08/12 CHECKED BY: R. Nashed	FIELD INVESTIGATION BY: M. Gaffney A. Kaddoura				

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 3660
PROJECT NUMBER & PHASE: 04120000041 CONTRACT NO.: 04-3G1601

DISREGARD PRINTS BEARING EARLIER REVISION DATES | REVISION DATES | SHEET | OF

FILE => 43G160qa02.dgn

USERNAME => S110822 DATE PLOTTED => 24-MAY-2013 TIME PLOTTED => 14:13

Memorandum

Flex your power!
Be energy efficient!

To: MR. HOOSHMAND NIKOUI
Branch Chief, Section A
Office of Geotechnical Design - West
Geotechnical Services
Division of Engineering Services

Attention: Mr. Ali Kaddoura

Date: April 12, 2012

File: 04-CC-24-PM 5.2
04-3G1601
Tieback Wall
Efis-0412000004

From: HOSSAIN SALIMI
Senior Materials and Research Engineer
Division of Engineering Services
Geotechnical Services - MS 5
Office of Geotechnical Design – West

Subject: Seismic Design Recommendations

This memorandum is in response to your request dated April 10, 2012 and presents the seismic design recommendations for the proposed Tieback Retaining Wall near Happy Valley Undercrossing, which is located about 3 miles west of the Route 24/680 Interchange in the Town of Lafayette in Contra Costa County.

According to the draft report submitted by Mr. Kaddoura, afield investigation was completed for this site by the Office of Geotechnical Design-West staff in March 2012. Based on the aforementioned report, " *Three power borings (R-12-001 through R-12-003) were drilled (March 2012) utilizing the rotary wash drilling method with Standard Penetration Test (SPT) sampling within the project limits. R-12-001 and R-12-002 were drilled to the depths of 91.5 and 76.5, respectively in the eastbound shoulder of Route 24 and R-12-003 was drilled to the depth of 51.5' in the inside median of eastbound Route 24. Borings R-12-001 and R-12-002 describe the foundation soils/rocks as approximately 35 to 40' of medium stiff to hard clays with gravel and some sand. This overlies about 5 to 10' of medium dense clayey sands and gravel. m of firm sandy clays with gravel. Hard silt lense was encountered in boring R-12-001 between the depths of 65' and 80' below roadway surface. The remainders of the borings describe the foundation soils/rocks as soft, very intensely to intensely weathered shale. Boring R-12-003 describes the foundation soils at the location of the tiebacks as about 20 of stiff to very stiff clays. This overlies about 10' of stiff to very stiff sandy silt with gravel. The remainder of the boring describes the foundation soils/rocks as soft very intensely weathered shale. The unconfined compressive strength of the clayey soils (using a pocket penetrometer) was estimated to range between 1.0 and 4.5 tsf. The SPT blow counts range from 5 to more than 50 (refusal) blows per foot. Refer to the Log of Test Boring (LOTB) sheets for details The LOTB sheets should be included with the contract plans and will be forwarded to you upon completion.*"

Based on the latest Caltrans ARS On-line map (V1.0.4), the closest faults to the site are Southampton Fault with Maximum Moment Mmax=6.3, located about 4.5 kilometers northeast of the bridge, Calaveras Fault with Maximum Moment Mmax=7.4, located less than 4 kilometer southeast of the bridge, and Hayward Fault with Maximum Moment Mmax=7.3, located over ten kilometers southwest of the bridge.

A Vs30=400 m/s was chosen for the site. The Acceleration Response Spectrum (ARS) curves based on both the new Deterministic Seismic Hazard Analysis (DSHA) and Probabilistic Seismic Hazard Analysis (PSHA) using a 975-year return period (5% probability of exceedance in 50 years) were generated for the site incorporating the latest Attenuation Relationship models. In addition, the PSHA with a 975-year return period using the USGS Interactive Deaggregation procedure was generated, and all four curves were compared. Due to the high seismicity of the site, the PSHA response spectra were both higher than the deterministic spectra (please see Figure 1).

Furthermore, the spectrum generated using the USGS Interactive Deaggregation procedure yielded higher amplitudes between 0 to 0.6 seconds and 1.1 to 1.9 seconds, whereas the spectrum generated using the Caltrans On-line procedure yielded higher amplitudes between 0.6 to 1.1 seconds and 1.9 to 4 seconds . Thus a composite of the two curves was generated to be used as the recommended final ARS curve (please see Figure 2).

In addition, the following table details the sources affecting the site, and the Peak Ground Accelerations generated from these sources.

SOURCE	Distance from project site (km)	Maximum Moment (Mmax)	Peak Ground Acceleration (g)
Calaveras Fault	3.6	7.4	0.37
Southampton	4.5	6.3	0.37
Probabilistic (Caltrans On-line)	N/A	N/A	0.65
Probabilistic (USGS Int. Deagg)	N/A	N/A	0.66

Please note that the final ARS curve has been modified to account for the proximity of the site to the faults. The modifications are such that there is no increase in spectral acceleration in periods less than 0.5 seconds and a 20% increase for periods greater than one second. A linear interpolation was used between 0.5 and one second.

Due to the nature of the materials encountered, the liquefaction potential at the site is considered minimal.

Mr. Hooshmand Nikoui
April 12, 2012
Page 3

If there are any questions, please contact Hossain Salimi at (916) 227-7147.

Attachments

c: TPokrywka (OGD-W)
MMacaranes (OGD-W)
Project file

REGISTERED PROFESSIONAL ENGINEER
HOSSAIN SALIMI
No. 54567
Exp. 12/2013
CIVIL
STATE OF CALIFORNIA

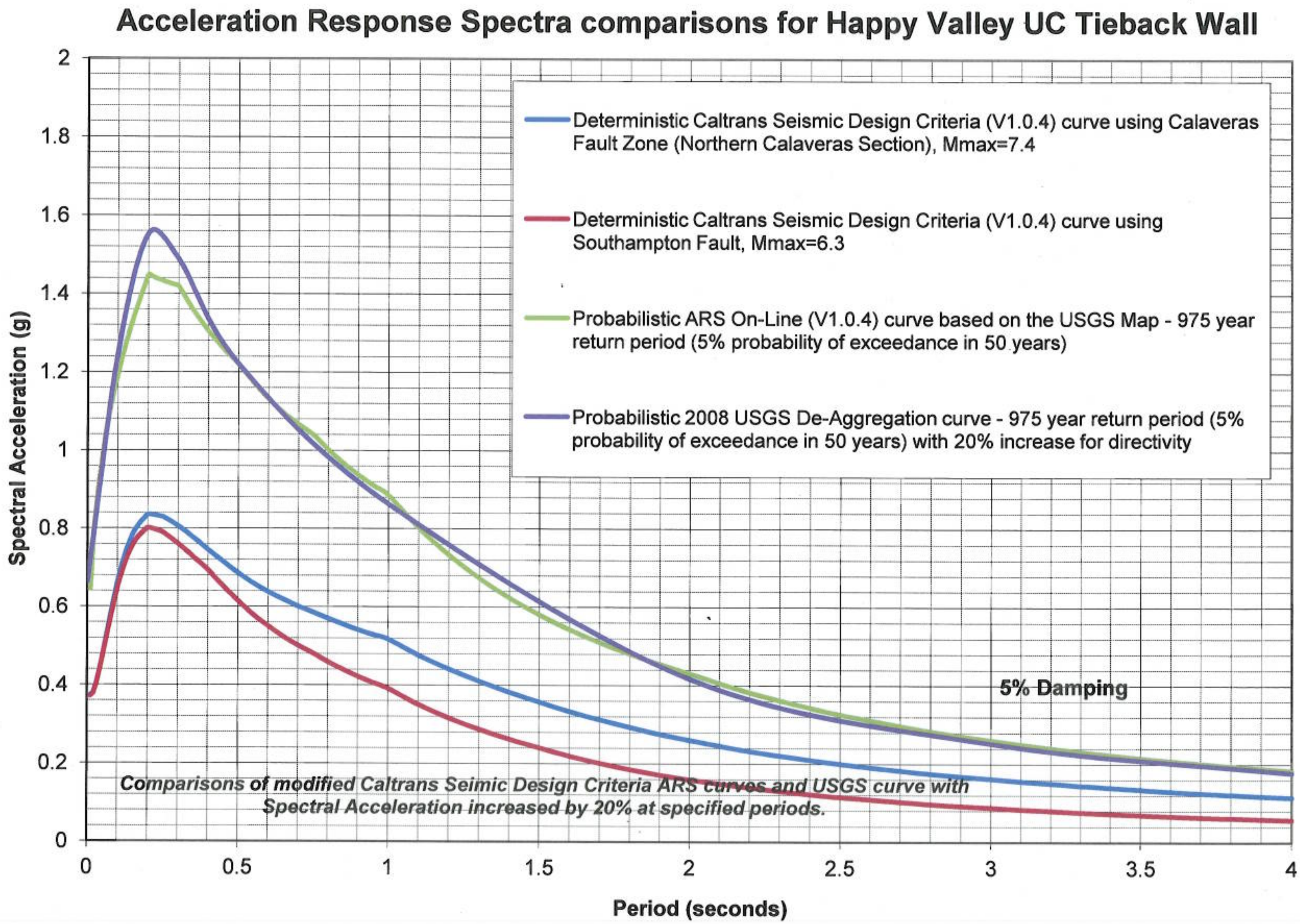

Acceleration Response Spectra comparisons for Happy Valley UC Tieback Wall
Spectral Acceleration (g)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Period (seconds)
0
0.5
1
1.5
2
2.5
3
3.5
4
Deterministic Caltrans Seismic Design Criteria (V1.0.4) curve using Calaveras Fault Zone (Northern Calaveras Section), Mmax=7.4
Deterministic Caltrans Seismic Design Criteria (V1.0.4) curve using Southampton Fault, Mmax=6.3
Probabilistic ARS On-Line (V1.0.4) curve based on the USGS Map - 975 year return period (5% probability of exceedance in 50 years)
Probabilistic 2008 USGS De-Aggregation curve - 975 year return period (5% probability of exceedance in 50 years) with 20% increase for directivity
5% Damping
Comparisons of modified Caltrans Seimic Design Criteria ARS curves and USGS curve with Spectral Acceleration increased by 20% at specified periods.

Figure 1

04-CC-24- PM 5.2
04-3G1601
Efis- 0412000004

Recommended Acceleration Response Spectrum for Happy Valley UC Tieback Wall

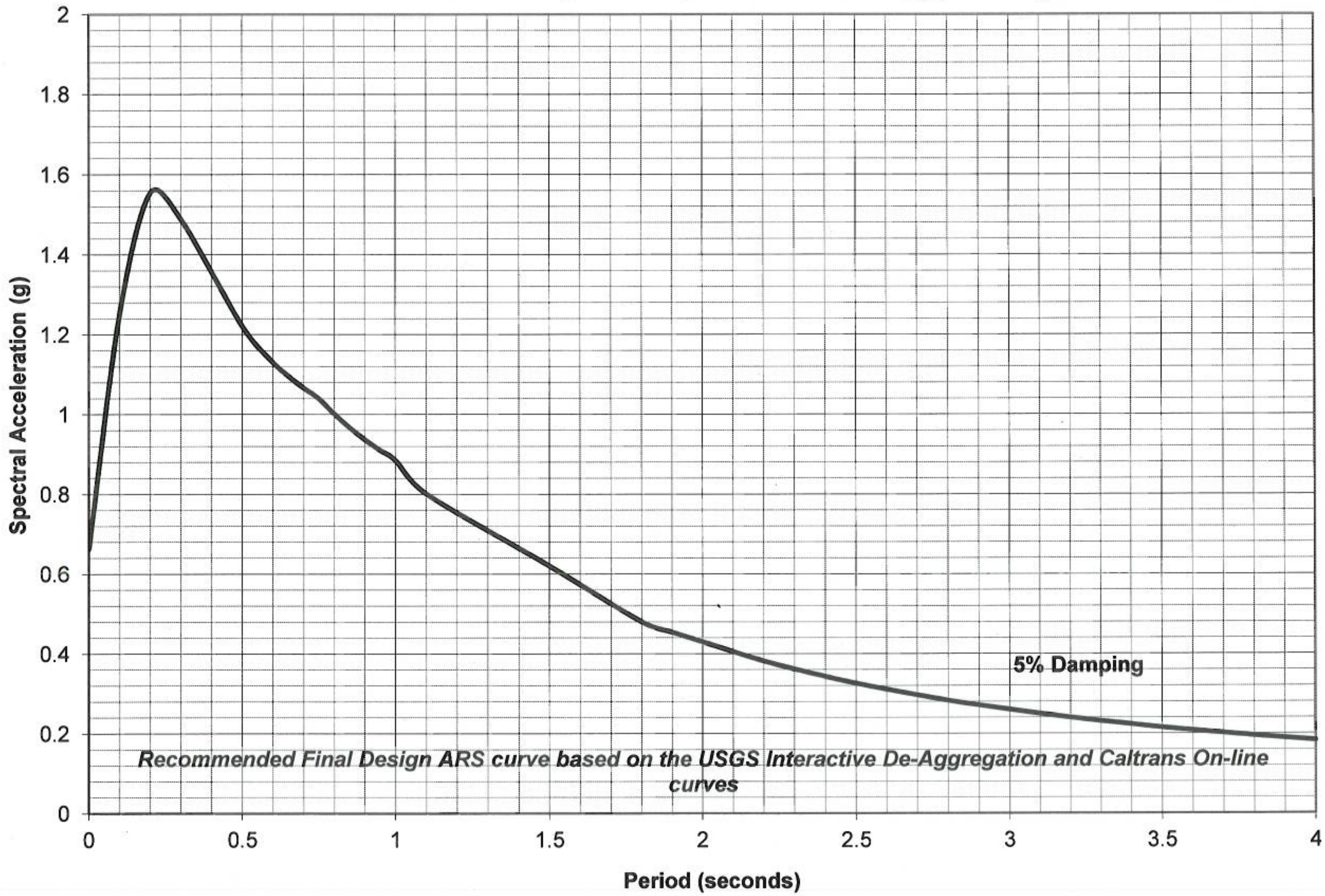
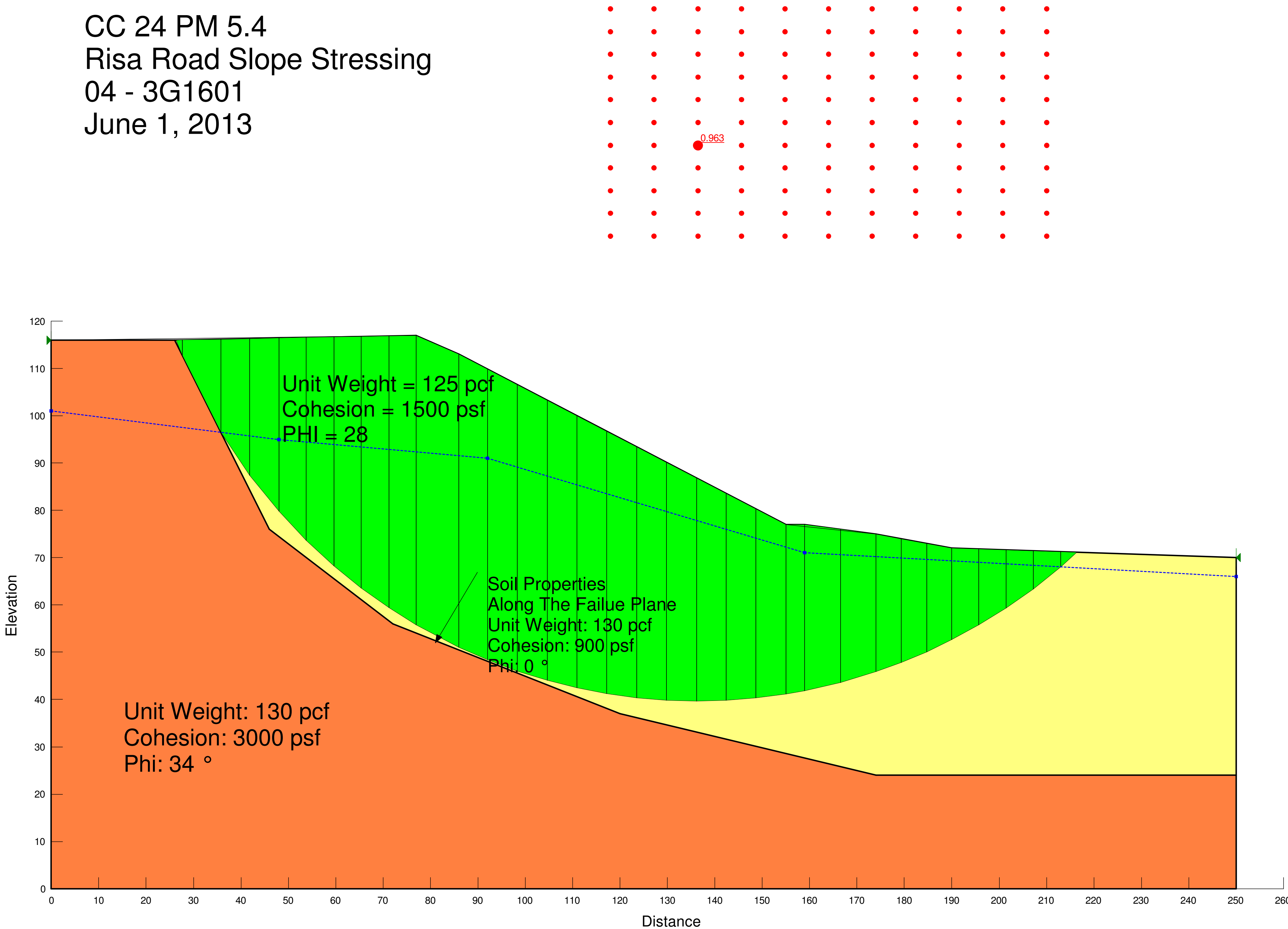
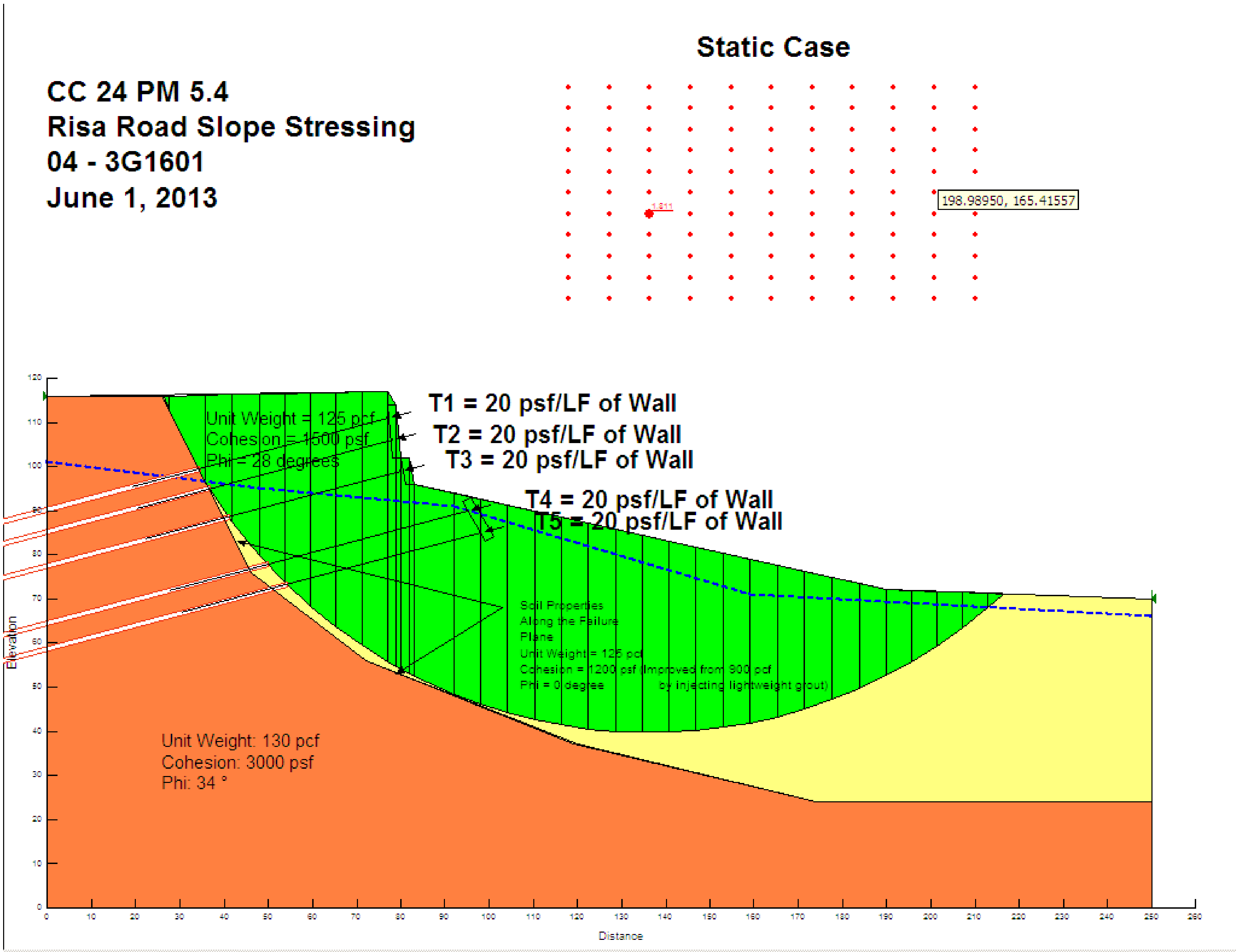
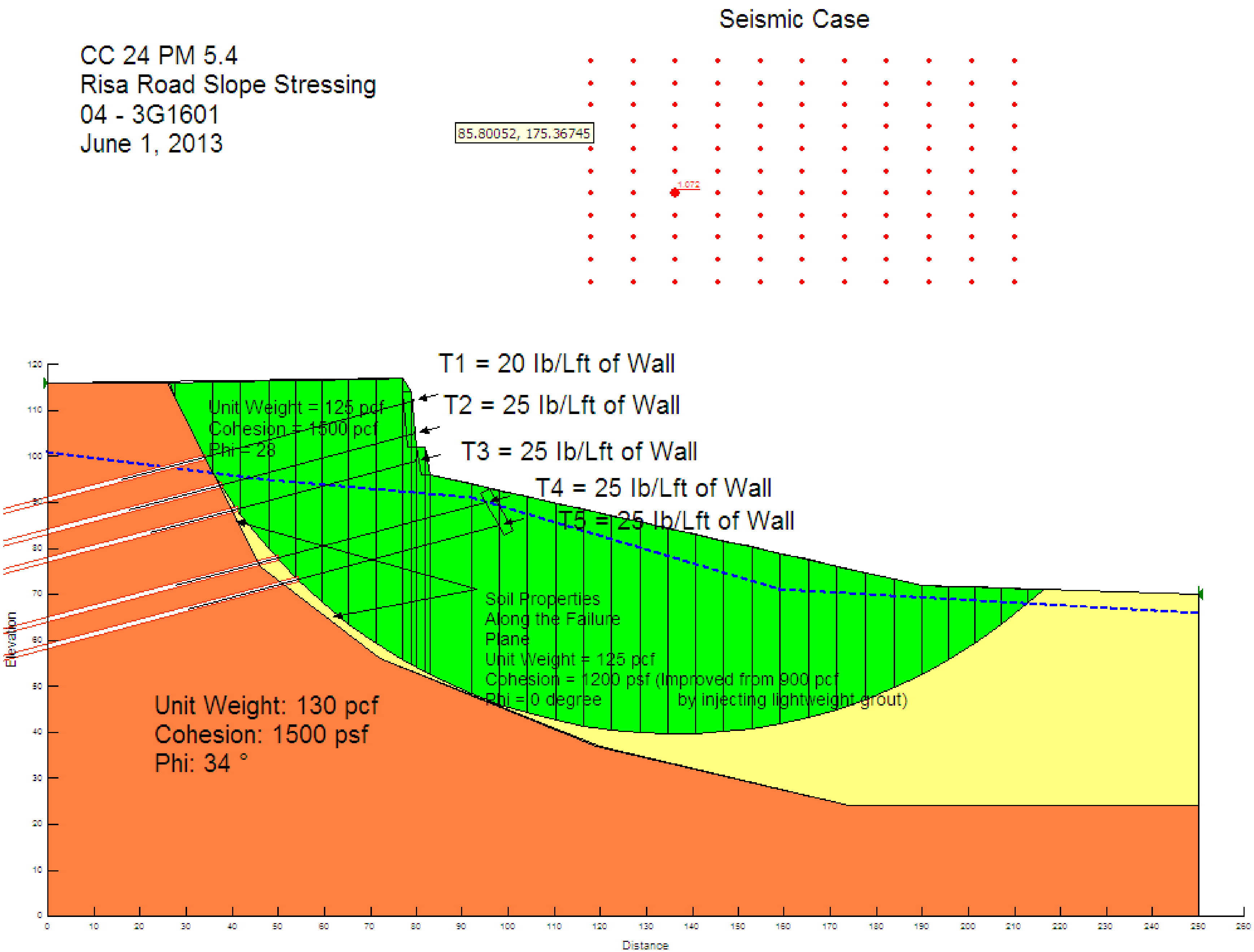


Figure 2

CC 24 PM 5.4
Risa Road Slope Stressing
04 - 3G1601
June 1, 2013




Static Case
CC 24 PM 5.4
Risa Road Slope Stressing
04 - 3G1601
June 1, 2013
198.98950, 165.41557
T1 = 20 psf/LF of Wall
T2 = 20 psf/LF of Wall
T3 = 20 psf/LF of Wall
T4 = 20 psf/LF of Wall
T5 = 20 psf/LF of Wall
Unit Weight = 125 pcf
Cohesion = 1500 psf
Phi = 28 degrees
Soil Properties
Along the Failure
Plane
Unit Weight = 125 pcf
Cohesion = 1200 psf (Improved from 900 pcf
Phi = 0 degree
by injecting lightweight grout)
Unit Weight: 130 pcf
Cohesion: 3000 psf
Phi: 34 °
Elevation
Distance


Seismic Case
CC 24 PM 5.4
Risa Road Slope Stressing
04 - 3G1601
June 1, 2013
85.80052, 175.36745
1.072
T1 = 20 lb/Lft of Wall
T2 = 25 lb/Lft of Wall
T3 = 25 lb/Lft of Wall
T4 = 25 lb/Lft of Wall
T5 = 25 lb/Lft of Wall
Unit Weight = 125 pcf
Cohesion = 1500 pcf
Phi = 28
Soil Properties
Along the Failure
Plane
Unit Weight = 125 pcf
Cohesion = 1200 psf (Improved from 900 pcf
Phi = 0 degree
by injecting lightweight grout)
Unit Weight: 130 pcf
Cohesion: 1500 psf
Phi: 34 °
Elevation
Distance