# INFORMATION HANDOUT

**For Contract No. 04-2G8904**
**At 04-Mrn-1-8.0**

**Identified by**
**Project ID 0400021259**

## MATERIALS INFORMATION

1. Foundation Report
2. Soil ADL and NOA Sampling Report

# 1. Foundation Report

**(March 08, 2016)**

**Marin County, Highway 1, PM 8.0**

**Project ID-0400025291/ EA-04-2G8904**

**Storm Damage Repair Project**

State of California
**DEPARTMENT OF TRANSPORTATION**

California State Transportation Agency

# Memorandum

***Serious drought***
***Help Save Water!***

**To:** MS. KELLY HOLDEN
Supervising Bridge Engineer
Office of Bridge Design - West

**Date:** March 08, 2016

**Attention:** Minh Ha

**File:** 04-MRN-1 PM 8.05
04-2G8901
EFIS# 0400021259
East of Slide Ranch

**From:** TUNG NGUYEN
Transportation Engineer
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

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Engineering Geologist
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CHRIS RISDEN
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**Subject:** **FOUNDATION REPORT (FR)**

## I. INTRODUCTION

This Foundation Report (FR) supersedes all our previous recommendations to mitigate a landslide located on north and southbound of the State Route 1, PM 8.05, close to Muir Beach, Marin County (see Appendix A). This revision was made based on the latest request by the Project Manager in his email of October 1, 2015 to keep the project construction within State-Right-of-Way because GGNRA would not issue permit for any easement outside of State-Right-of-Way. The revision made to the original design for accommodating this limitation included reducing the depth of excavation in front of the wall, reducing the number or eliminating the ground anchors, and use of stiffer soldier beams without ground anchor, where feasible.

The Office of Geotechnical Design – West has investigated the landslide at the above mentioned location based on the available subsurface soils/rock information, site visits and the new restriction to be inside of the R/W. During our recent site visits, we performed visual reconnaissance of the subject area to assess the nature and extent of the reported problem. This memo presents our observations, investigation, our repair method and the associated foundation design parameters and construction considerations.

## II. BACKGROUND

During the Winter of 1982-1983, severe storms saturated and weakened the slopes at this location. Slipout and drop-offs up to 15 feet deep developed at the edge of the road and settlement of several feet occurred within the traveled way. The repair included realigning Highway 1 to the east by cutting the upslope during 1983. Cut slopes of 1.5H:1V (Horizontal to Vertical) were done from PM 8.0 to PM 8.15 and metal underdrain pipes were installed.

During 1986, horizontal drains were installed at PM 8.03 to reduce the groundwater elevation and increase the slope stability. During the Winter 2003-2004, we observed cracks that extended from the shoulder of the down slope side (southbound) to the shoulder of the upslope side (northbound) of Highway 1, at approximately Post Mile 8.05. The overall length of road where cracking observed was approximately 300-feet. During 2006, we installed one SI to monitor the movement of the slide and our data revealed a deep seated slide of approximately 30 feet deep below the roadway grade.

During our 2011 site visits, we noted cracks up to 2 inches wide and numerous faulting up to 2 inches on almost entire area of travel way. We installed an additional SI and one piezometer on March 2014 in order to determine the current depth of the slide and continue the slope movements monitoring.

## III. PHYSICAL SETTINGS

### III.1 Climate

The project lies in coastal Marin County, between Muir Beach and Stinson Beach. The climate is moderate, with cool, rainy winters and mild, dry summers. Heavy fog is common. Most rain falls in the winter months, with the peak rainfall typically in January or February.

### III.2 Topography and Drainage

Highway 1 at this location traverses a steep hillside. The slope above the repair exposes rock, and groundwater drips freely from the slope face. At the repair site, the slope above the roadways drops off sharply, but is stable. Adjacent slopes are stable, without slides or slumps or erosion features.

## IV. GEOLOGY

### IV.1 Regional

Marin County lies in the California Coast Ranges, a northwest-trending band of folded and faulted mountains that roughly parallel the San Andreas fault zone. The steep hills at the site are made up of Cretaceous Franciscan Complex.

The region is highly seismically active, with numerous active and potentially active faults. The San Andreas Fault lies 2.2 mi (3.59 km) from the site.

## IV.2 Site

The site lies on the Cretaceous Franciscan Complex, mélange terrane[1]. The mélange terrane is described as chaotic mixtures of fragmented rock masses in a sheared matrix, or block-in-matrix rock. The slope above the culvert is argillite, a pervasively sheared mudstone, with larger blocks of harder rock up to 3.28 ft (1 m) across.

## IV.3 Excavation and Drilling Characteristics

Franciscan bedrock is known as "block-in-matrix" rock. This means that resistant blocks of bedrock are randomly distributed in a highly sheared, weaker matrix. The blocks may be as small as fraction of an inch (centimeter) scale to tens of feet (meters) across. Normal stratigraphic relations (layer-cake geology) do not apply to block-in matrix rock, as blocks are random in size and in distribution.

Drilling and excavating conditions may be highly variable at the site, and because of the random distribution of blocks, test borings are not necessarily representative of whole site. While the borings show argillite, highly resistant blocks of unknown size may be encountered in excavation.

Estimating the rock type percentage from the boring logs and the exposed cut slope above the repair site, it appears that most of the material will be pervasively sheared shale logged in the borings, with larger hard blocks of up to 3.28 ft (1 m) diameter. The cut slope above the site consists of approximately 10% large greenstone blocks about 3.28 ft (1 m) in diameter, up to 6.56 ft (2 m) and 90% argillite matrix. The contractor should develop excavation plans suitable for the site conditions accordingly.

## IV.4 Seismicity

Table 1 below lists the major faults in the region, their distance from the project site, and peak bedrock accelerations anticipated at the site obtained with Caltrans ARS Online v2.3.06 (Tools to Establish Design Seismic Hazard).

**Table 1: Faults - Peak Bedrock Accelerations**

| Fault | Distance, mi (km) | Peak Bed Rock Acceleration, g |
|---|---|---|
| San Andreas | 2.2 (3.59) | 0.48 |
| San Gregorio | 3.6 (5.80) | 0.39 |
| Hayward | 14.2 (22.94) | 0.15 |

[1] Wagner, D.L., Bortugno, E.J., and McJunkin, R.D., 1990, Geologic Map of the San Francisco Quadrangle, California Division of Mines and Geology, 1990, scale 1:250,000.

## V. FIELD INVESTIGATION

### V.1 EXPLORATION

Two borings (R-14-001 and R-14-002) were drilled during March 2014 utilizing the rotatory wash drilling method to a depth of 75 and 55 ft, respectively below the roadway elevation.

The logs of test borings which drilled in March 2014 are shown in Appendix B.

The borings were drilled on the southbound lane immediately inside the slipout headscarp. Soil/rock samples were collected from these borings for observation and assessment. The rock cores retrieved from the borings are stored in the District 4 Lab in San Francisco and they can be made available for review upon request.

### V.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered in the 2014 borings show that the site has about 5 ft of pavement (asphalt plus base) over Franciscan Formation (Franciscan Mélange). The Franciscan Formation encountered in boreholes R-14-001 and R-14-002 consisted of fine to medium Graywake; massive; fresh to intensely weathered; hard to very hard; slightly to intensely fractured; matrix is lean clay with sand; soft to stiff; brown gray and greenish gray; poor cemented.

The thickness of the shale and graywake could vary along the proposed wall alignment. Due to nature of the rock at this site as mentioned above, blocks of hard rock will be encountered during the proposed piles, ground anchor wall and soldier pile wall construction.

Groundwater at the site varies depending on the season and it could fluctuate from about 5 ft to about 15 ft depth below the existing ground surface.

### V.3 INSTRUMENTATION

During 2006 we installed a slope inclinometer labeled SI-8 to a depth of 48 ft. The measured cumulative displacement to January 2014 is 1.9 inches and the measured depth of the slide displacement below the roadway grade is about 26 feet. These data are confirmed by the newly installed SI in 2014. These SI data and plots are presented in Appendix C.

## VI. SOIL/ROCK GEOTECHNICAL PARAMETERS

Soil/rock strength parameters of the sliding mass were determined using back-analysis of the landslide. This is necessary because it is impractical 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, head scarp location, and other geologic features. Our back analysis shows that slide soil material in dry condition has an effective friction angle of 16° and cohesion of 0 psf (along the slip-plane) for safety factor (SF) of 1.0. To simulate the faster landslide movement during the wet periods, we increased pore water pressure in the sliding mass to reduce

the SF to 0.95. The soil strength determined by back-analysis and assumed pore water pressure parameters are summarized in Table 2.

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

**Table 2: Back Calculated Soil/Rock Strength and Pore Water Pressure**

| Soil/Rock Type | Unit Weight, psf | Internal Friction Angle $\phi$, degrees | Cohesion c, psf | Pore Water Pressure Parameter $r_u$ | Safety Factor (SF) |
|---|---|---|---|---|---|
| Fill/Franciscan Mélange | 130 | 16 | 0 | 0 | ~ 1.00 |
| Fill/Franciscan Mélange | 130 | 16 | 0 | 0.08 | ~ 0.95 |

## VII. RECOMMENDATIONS

The most viable repair strategy for this location is to construct about 53 ft long soldier pile retaining wall system from Sta. 403+97 to Sta. 404+50, and about 334 ft long soldier pile with ground anchors wall from Sta. 404+50 to Sta. 407+84. The ground anchor wall section will be approximately 26 feet and the soldier pile section 12 ft in height for the long term condition. Since the anchored wall is located on the downhill slope it requires placement of backfill. Soldier pile wall section is buried and doesn't need excavation in front of it.

### VII.1 GROUND ANCHOR WALL

We performed additional slope stability analyses for our proposed repair system using a specified slip-plane and back calculated soil/rock parameters, and pore water pressure condition. The analyses were performed to determine the ground anchor loads and resulting SF. The results of the stability analysis are attached in Appendix D.

Below are summary of soil/rock strength parameters, design ground anchor loads and soldier piles parameters.

**Table 3: Rock/Soil strength parameter**

| Rock Type | Unit Weight ($\gamma$) pcf | Friction angle ($\phi$) Degree | Cohesion (c) psf |
|---|---|---|---|
| Slide Rock | 130 | 16 | 0 |
| Foundation Rock | 135 | 32 | 0 |

**Table 4: Ground anchor Loadings**

| Type of Loading | SF | Ground Anchor Level 1 Load On Retaining Wall | Ground Anchor Level 2 Load On Retaining Wall |
|---|---|---|---|
| Static | >1.5 | 14 Kips/ft @ 15˚ angle | 14 Kips/ft @ 25˚ angle |
| Seismic | > 1.1 | 22 Kips/ft @ 15˚ angle | 22 Kips/ft @ 25˚ angle |

**Table 5: Soldier Pile for Ground Anchor Wall**

| Wall Height, ft | Pile Spacing, ft | Minimum Pile Diameter, ft | Total Pile Length, ft |
|---|---|---|---|
| 13 | 5 to 7 | 2 | 41 |

The friction and tip bearing capacities in compression of these soldier piles are presented in Table 6.

**Table 6: Pile Friction and Tip Compression Capacities**

| Pile Capacity | Ultimate, ksf | Allowable, ksf |
|---|---|---|
| Friction Capacity Below The Dredge Line Of The Wall | 1.44 | 0.72 (SF=2) |
| Tip Compression Bearing Capacity | 68.0 | 22 (SF=3) |

The proposed soldier piles ground anchor retaining wall system can be designed using the earth pressures and criteria outline in August 2004 Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.7 Figure 5.5.5.7.1-1a, which are included in Appendix E. Note that this design lateral earth pressure is 8% higher than the AASHTO LRFD BDS Six Edition earth pressure presented in the Section 3.11.5.7.1.

- For earth pressure against the wall, layer 1 above the failure, down to depth of 26 ft use internal friction angle $\phi$ =16°, cohesion c = 0 psf, and total unit weight of soil/rock $\gamma$ = 130 pcf. Use active earth pressure coefficient (Ka) of 0.647. Use passive earth coefficient (Kp) of 1.15 for soil in front of the wall from bottom of excavation to the slide failure plane.

- For earth pressure against the soldier piles, layer 2 below the failure plane, use engineering properties of Franciscan Melange; internal friction angle $\phi$ =32°, cohesion c = 0 psf, and total

unit weight of soil/rock $\gamma$ = 135 pcf. Use active earth pressure coefficient (Ka) of 0.31 and passive earth coefficient (Kp) of 3.25.

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

The above recommended design parameters 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.

Add traffic load equivalent to a rectangular pressure diagram equivalent to 2 ft of fill applying from the top of the wall to a depth equal to the wall height.

The wall shall be capable of resisting an additional seismic earth pressure estimated to be equal to 34H psf (max) with triangular distribution similar to active soil pressure, which is applied at H/3 from the bottom of the wall.

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

Based on the above, we recommend using Design Pullout Load of 14 Kips/ft and a Max Pullout Test Load of 22 kips/ft of the wall. Due to the large size of the failure the safety factor of 1.3 is not adequate and we had to use a safety factor of 1.5.

The above recommended design parameters 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 the following additional requirements for the ground anchors:

- The proposed first row of ground anchors should be installed at 5 ft below the existing ground surface and they should be installed at an angle of 15 degrees below the horizontal. The second row of ground anchors should be installed 5 ft below the first row at an angle of 25 degrees below horizontal.

- Ground anchors shall conform to specification in Chapter 46 of the 2010 Standard Specifications.

- The unbonded lengths of the 1st and 2nd ground anchor rows should be 35 ft and 26 ft long, respectively.

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

- Pile spacing should be limited to no more than 7 ft.

- The ultimate and allowable vertical compression capacities of piles are specified in Table 6.

- Use 50 percent of the compression shaft resistance values mentioned above to calculate the ultimate tension (uplift) resistance of the pile.

- Wood lagging shall extend at least to the bottom of the bench estimated at about 13 ft depth below the existing ground surface. The bench width should be at least 8 ft.

The District 4 Design shall check and inform us that we do not exceed the R/W limits by following the above recommendations. Otherwise, we will make adjustment to the above parameters.

## VII.2 SOLDIER PILE WALL

Below are our recommendations for the wall structure design requirements:

The summary of soil/rock strength parameters is shown above on Table 3.

Soldier pile wall requirements are shown on Table 7. No benching and wood lagging are required for the soldier beam wall section.

**Table 7: Soldier Pile Wall**

| Wall Height, ft | Pile Spacing, ft | Minimum Pile Diameter, ft | Total Pile Length, ft |
|---|---|---|---|
| 12 | 5 | 2 | 27 |

Pile friction and tip compression capacities for 2 ft diameter pile with min embedment of 15 ft is shown above in table 6.

The proposed soldier pile retaining wall system can be designed using the earth pressures given above in Section VII.1 and criteria outline in Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.6 Figure 5.5.5.6-1.

Add traffic load equivalent to a rectangular pressure diagram equivalent to 2 ft of fill applying from the top of the wall to a depth equal to the wall height.

The wall shall be capable of resisting an additional seismic earth pressure estimated to be equal to 34H psf (max) with triangular distribution similar to active soil pressure, which is applied at H/3 from the bottom of the wall.

We recommend the following additional requirements for the soldier pile wall:

- Pile spacing should be limited to not more than 5 ft.

- The ultimate and allowable vertical compression capacities of piles are specified in Table 6.

- Use 50 percent of the compression shaft resistance values mentioned above to calculate the ultimate tension (uplift) resistance of the pile.

## VII.3 BACKFILL

In order to provide a drained condition conforming with the design parameters indicated above, the wall backfill shall be structure backfill conforming with those specified for metallic soil reinforcement in section 47-2.02C of the Standard Specifications. The backfill shall be compacted to 95% relative compaction using light to medium compaction equipment.

## VII.4 EMBANKMENT CONFINEMENT SYSTEM (ECS)

We recommend installing ECS on the bench in front of the ground anchored wall to cover the wall face as required for aesthetics by permitting agencies. The ECS facing is inclined about 60 degrees toward the wall. The back fill consists of a narrow topsoil behind the face to allow vegetation growth followed by regular compact structure fill. The ECS is constructed in multi lifts of about 2.3 ft thick. The top of the ECS embankment shall conform to the desired profile. See Appendix F for details. Soil subgrade which receives the ECS shall be prepared to grades and line shown on the plans and be free from organic and loose soil and rock zones and compacted to minimum 95% relative density with light to medium weight compaction equipment.

The minimum ECS reinforcement length shall be 0.7H, where H is the exposed height of the wall to be covered with the fill. If this cannot be met due to combination of insufficient bench width, ECS face slope angle, and the wall height, vertical gabion baskets with a total height of no more than 4 ft can be placed on the excavated prepared bench level below the ECS embankment. Please refer to gabion basket details on sheet D100A and D100B of 2010 Standard Plans. If the reinforcement requirement still cannot be met with gabion inclusion, the end of reinforcement shall be fixed to the face of wall. Structure Design is required to develop detail to achieve this. Geotechnical design west has used two different details for Director Order Projects that can provide to SD if needed. If this option is used the ground anchor Design and Maximum Test Load shall be increased by 0.6 Kips/ft and 0.9 Kips/ft, respectively.

The bench in front of the wall where the ECS is going to be installed shall slope 2% toward the face of the wall. A backfill of crushed Class 3 Permeable Material 0.5 to 1.5 ft thick wrapped on filter fabric shall be placed on the top of the bench prior to installing the gabion or ECS (see Attachment F). The ECS will be backfilled with Class 2 Permeable Material (crushed) and in the face fill with compact Top Soil. The permeable material shall conform with Section 19-3.02C of the 2010 Standard Specifications. Permeable Backfill used for ECS shall be compacted to 90%.

## VIII. DRAINAGE

We recommend installing horizontal drains of minimum 1.5 inches in diameters at maximum spacing of 15 ft at variable elevations defined in plans. The horizontal drains shall be about 45 ft in length and installed at about 2 degree above the horizontal plane.

All horizontal drains shall be discharged into a minimum 6 inch solid pipe clamped to the base of the wall face. The pipe can be discharges at few locations, as needed, along its length or at both its ends depending on the pipe profile and its length. Please consult with District 4 Hydraulics Branch For horizontal drains discharge collection system and its outlet and use of rock dissipater at the outlets as well as for surface run off drainage and collection system. In the case of restoring moisture in ECS backfill for planting, the other alternative of discharge is recommended such that all horizontal drains shall be discharged into a minimum 6-inch wide horizontal geocomposite or collector drains laid on compacted backfill of ECS in front of soldier pile wall.

## IX. CORROSION

No corrosion testing was performed. Based on our experience of similar projects constructed near this site we recommend the use of Marine Atmosphere protection measures for this project in accordance with Section 8-38, Table 8.22.1 (Minimum Concrete Cover for 75-year Design Life) of the September 2003 Caltrans Bridge Design Specifications.

## X SHOTCRETE VERSUS WHALER OPTION

Soldier beam ground anchor walls traditionally are designed with use of wood lagging and stiff walers to bridge the adjacent beams and transfer all the lateral earth pressure to the beam and ground anchors. These systems performed very successfully over many years due to its rigidity, load transfer mechanism between the shallower and deeper than the slide depths that assumed in the design, and its drainage ability. The above recommendations are for the soldier beam ground anchor wall with wood lagging and waler beams system. If the use of shotcrete is proposed instead of the wood lagging and whaler, we recommend the following:

1) Use three rows of ground anchors, each with design load of 9.5 Kips/ft and maximum test load of 14 kips/ft with the same unbonded lengths of 35 ft for the 1$^{st}$ row and 26 ft for the 2$^{nd}$/3$^{rd}$ rows. This is to provide more uniform fixity between the shotcrete and the beams. The recommended ground anchor load shall be increased by the additional load due to ECS reinforcement pull out as described in Section VII-4.

2) Use structural shotcrete with a minimum thickness of 13.5 inches to provide same rigidity as that of the soldier beam ground anchor with wood lagging and waler beams wall system.

3) The structural shotcrete shall remained intact under the maximum earth pressure calculated for static pressure outline in Bridge Design Specifications (BDS) Section 5.5.5 Earth Pressure, Art 5.5.5.7 Figure 5.5.5.7.1-1b and that increased by a factor of 1.3 for the seismic loading conditions. In addition, the structural integrity of the entire beam and shotcrete system and shotcrete facing panels need to be checked by the designer for all of the excavation stages.

4) Both weep holes above the finished bench and the horizontal drains at a maximum spacing of 15 ft near the bottom the maximum excavation depth are required. Please refer to Drainage Section VIII in the above for collecting and discharging the horizontal drains.

## XI. CONSTRUCTION CONSIDERATIONS AND REQUIREMENTS

The following construction considerations and requirements should be included in the design and construction specifications for the proposed wall:

- The Contractor will encounter difficulties during drilling for the soldier beam pile, ground anchor holes and horizontal drains. This is likely due to hard drilling in large blocks of hard rock in a soft clay matrix of loose/soft slide mass and shale, and presence of high groundwater. Temporary casing of the drill hole and dewatering is likely required.

- During drilling operation for the proposed soldier beam piles, we believe that some caving of the drilled holes will likely occur. This also dictates the use of casing combined with dewatering.

- During the drilling operation for the proposed soldier beam piles, we believe that some caving of the drilled holes will likely occur. Thus, use of temporary casing is required. Due to the groundwater elevations, the installation of soldier piles will likely require dewatering of the borehole before the concrete is placed. The current Caltrans practice for soldier beam pile construction does not allow the use of slurry. Therefore, the use of temporary casing and dewatering is required when groundwater is encountered during construction of the soldier beam pile.

- The drilling and concrete placement for soldier beam piles construction shall be staggered. No two adjacent holes can be open at the same time. The drilled hole for the soldier beam piles can't be left open overnight.

- Installation of the soldier beam piles should be performed in accordance with Section 49-4 of the May 2010 Caltrans Standard Specifications.

- Due to a potential for undesired ground movement, all surcharge loads placed on the ground by heavy equipment shall be spread uniformly over the contact area and shall not exceed 400 psf. No stockpiling of soil and construction material more than three (3) feet is allowed.

- Any temporary back cuts during construction shall be no steeper than 1.0(H) to 1.0(V). All excavations shall follow Cal/OSHA excavation requirements.

- In order to keep construction works within R/W limits, temporary shoring may be required. The Contractor shall design and submit to Department for the approval.

- Hazardous waste should be assessed by Caltrans District 4 Hazardous Waste Branch.

- Earth materials as well as groundwater conditions can vary between the points of exploration and observations in type, properties, and strength. Therefore, we do not and cannot have full knowledge of the subsurface conditions underlying the site. The recommendations and conclusions presented in this FR are based on the findings of the points of exploration,

interpretation, and extrapolations of information between and beyond these points are subject to confirmation based on the conditions revealed during construction.

- Water that has infiltrated the drilled hole shall be removed before placing concrete therein. Fluvial or drainage water shall not be permitted to enter the hole.

- Backfill of a portion of the wall might be required. For this portion, structure fill shall be used and compacted in lifts not thicker than one foot to 95% relative compaction before placement of ground anchor. Light compaction equipment shall be used near the wall face.

The recommendations contained in this FR are based on specific project information regarding structure type and location. If any conceptual changes are made during final project design, the Office of Geotechnical Design-West, Design Branch C should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Tung Nguyen/Ron Karpowicz at 510-622-1775/510-286-5640 or Mahmood Momenzadeh/Chris Risden at 510-286-5732/510-622-8757.

Attachments:

Appendix A - Location Map
Appendix B - Boring Logs
Appendix C - SI Data and Plots
Appendix D - Back Analysis
Slope Stability
Appendix E - Lateral Earth Pressures
Appendix F - Embankment Confinement System (ECS)

c: WNyaz, BNguyen, RSchaerli, SGalvez, RFernandes, RChan, http://svgcgeodog.dot.ca.gov/, TPokrywka, MMomenzadeh, CRisden, RKarpowicz, Daily File

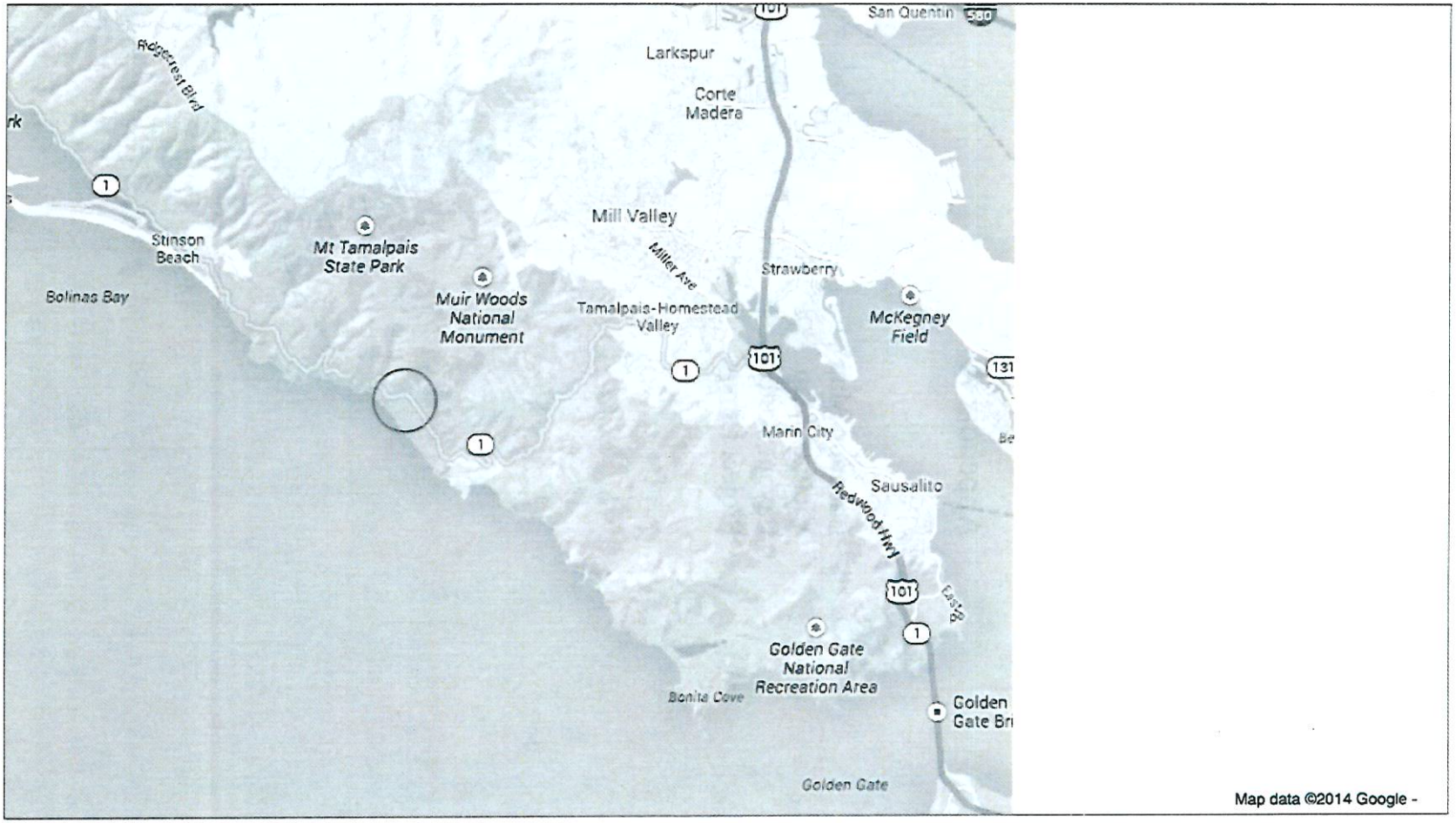
TNguyen/mm

# Attachments

# Appendix A
# Location Map

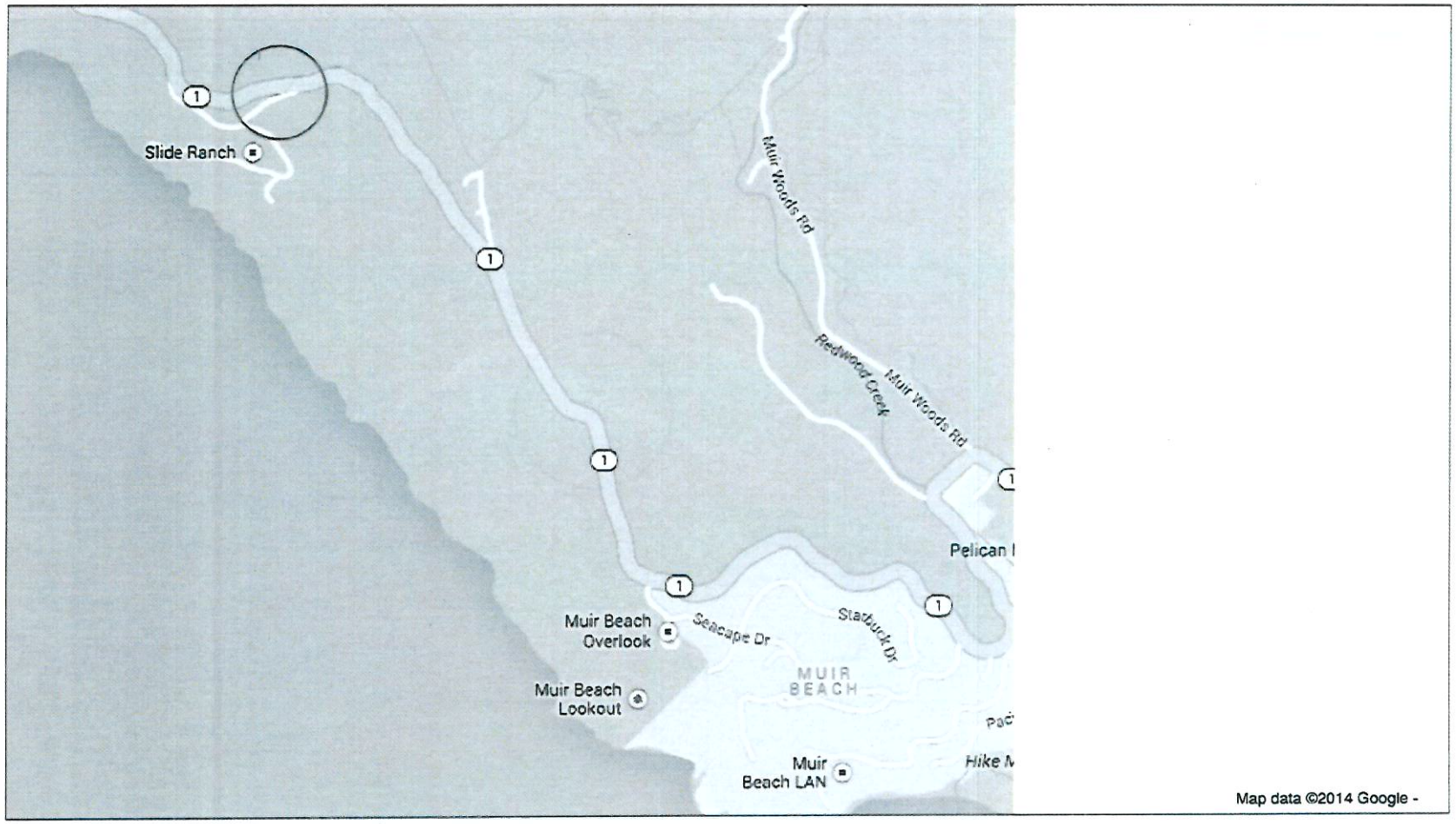
To see all the details that are visible on the screen, use the "Print" link next to the map.

Google



To see all the details that are visible on the screen, use the "Print" link next to the map.

Google



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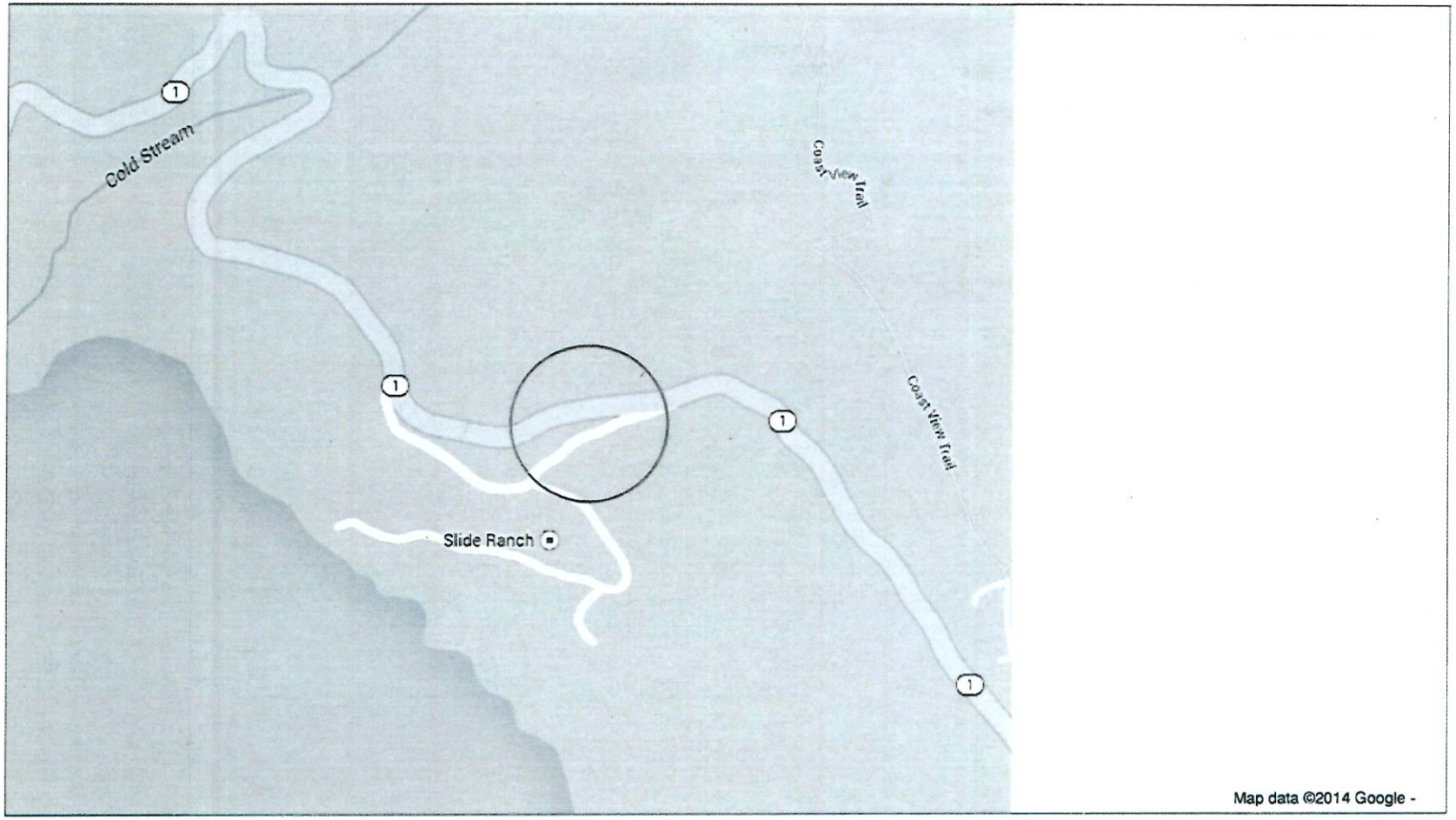
Google



VICINITY MAP
4-MRN-1 PM 8.05
APPROX. LIMITS
© 2011 Europa Technologies
Data SIO, NOAA, U.S. Navy, NGA, GEBCO
© 2011 Google
138 ft
Google
Pointer 37°52'30.09" N 122°35'49.56" W elev 305 ft Streaming 100% Eye

8 05

# Appendix B
# Boring Logs

| DIST | COUNTY | ROUTE | POST MILE TOTAL PROJECT | PREV. ANY/AB Rs | PREV. ANY/AB SHEETS |
|---|---|---|---|---|---|
| 04 | Mrn | 1 | 8.05 | | |

PROFESSIONAL GEOLOGIST 10-28-15

PLANS APPROVAL DATE

Chris Risden No. 7795 Exp. 9-30-17 STATE OF CALIFORNIA

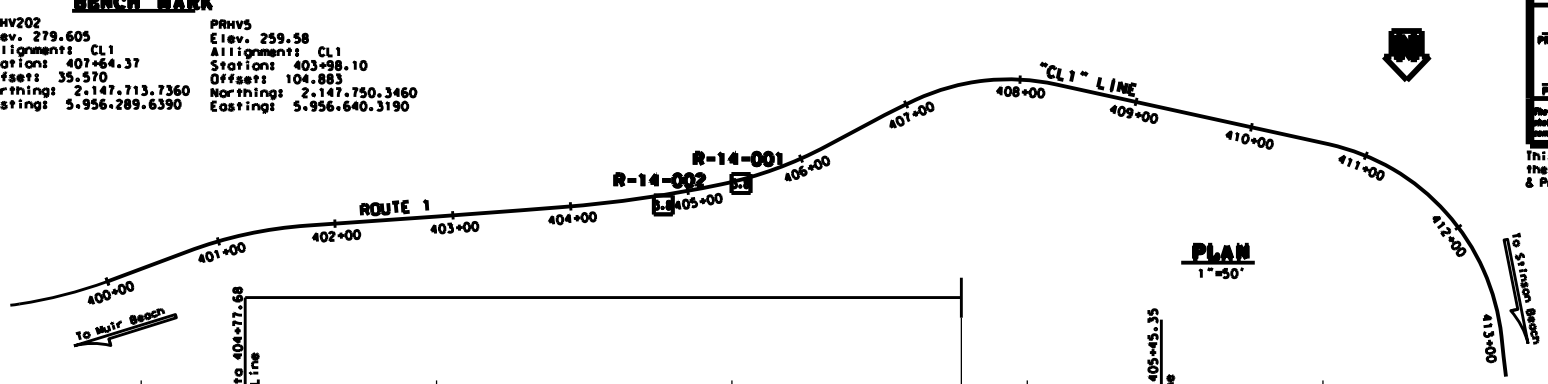
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

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

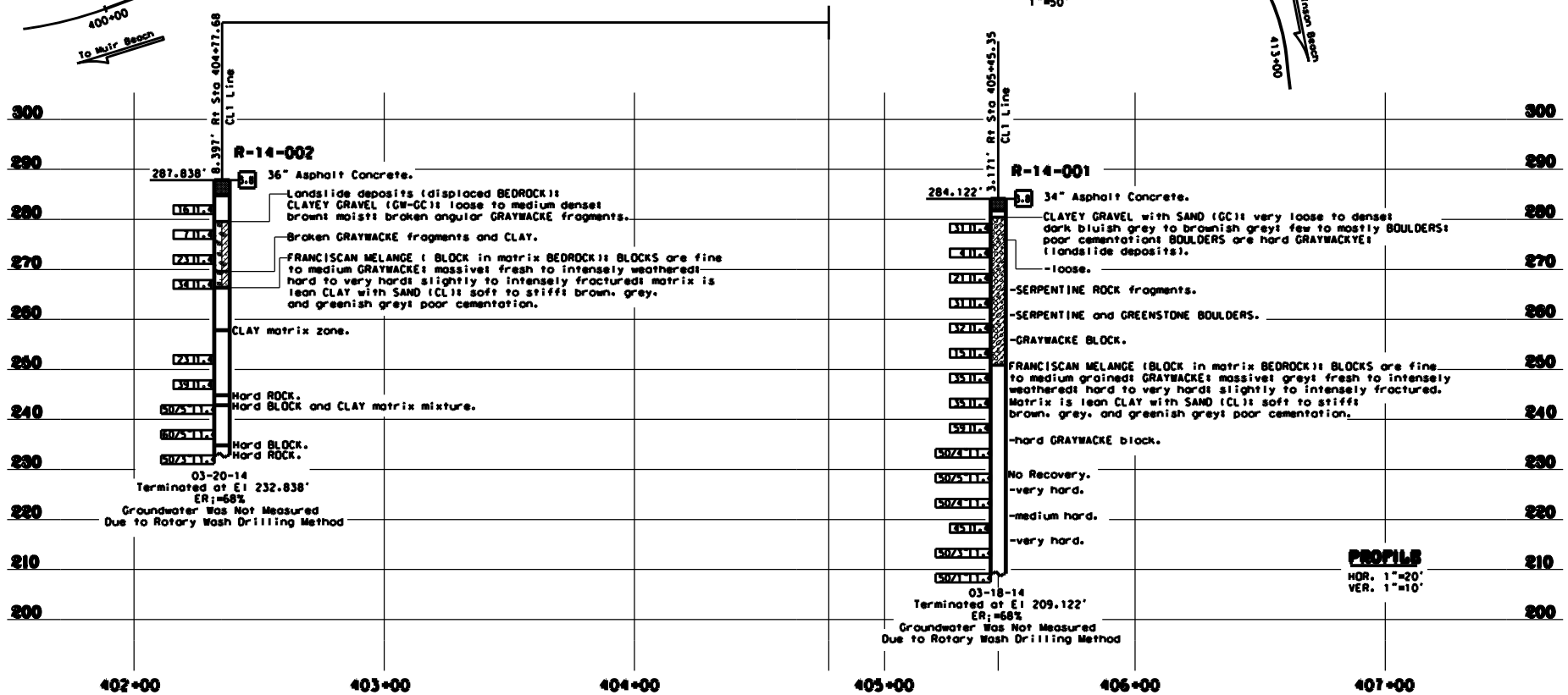
## BENCH MARK

PRHV202
Elev. 279.605
Allignment: CL1
Station: 407+64.37
Offset: 35.570
Northing: 2,147,713.7360
Easting: 5,956,289.6390

PRHV5
Elev. 259.58
Allignment: CL1
Station: 403+98.10
Offset: 104.883
Northing: 2,147,750.3460
Easting: 5,956,640.3190




## PLAN
1"=50'




## PROFILE
HOR. 1"=20'
VER. 1"=10'

| ENGINEERING SERVICES | GEOTECHNICAL SERVICES | STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION | DIVISION OF ENGINEERING SERVICES OFFICE OF GEOTECHNICAL DESIGN BRANCH | RETAINING WALL EAST OF SLIDE RANCH |
|---|---|---|---|---|
| FUNCTIONAL SUPERVISOR | DRAWN BY: N. Reynolds 08/14 | FIELD INVESTIGATION BY: R. Karpowicz | | LOG OF TEST BORINGS 1 of 1 |
| HAND: H. Nemensedeh | CHECKED BY: E. Ortega | | | |

SEE CIVIL LOG OF TEST BORING SHEET

DISPLAY SCALE IS SHOWN FOR ORIGINAL PLANS 0 1 2 3

UNIT: 3660
PROJECT NUMBER & PHASE: 04000212590 CONTRACT NO.: 04-2G8900

DESCRIBE POINTS DEALING WITH REVISION DATES

FILE -> $REQUEST

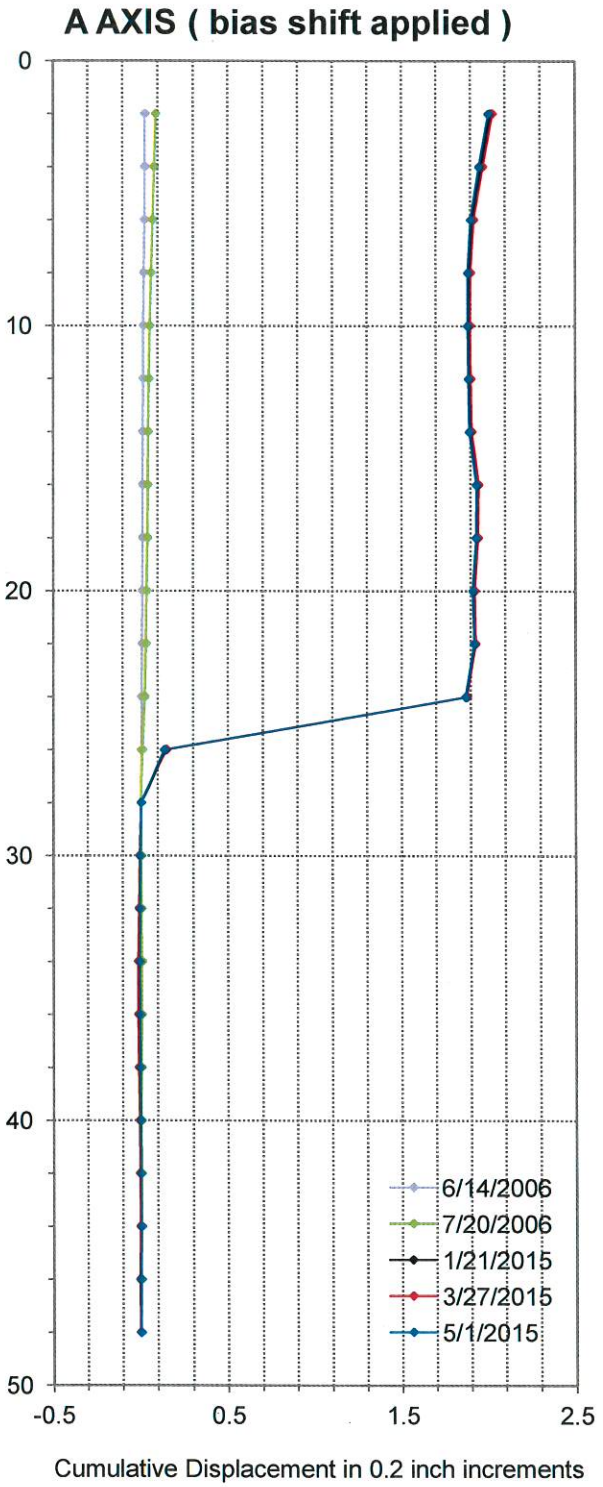
USERNAME: $USER DATE PLOTTED: $DATE TIME PLOTTED: $TIME

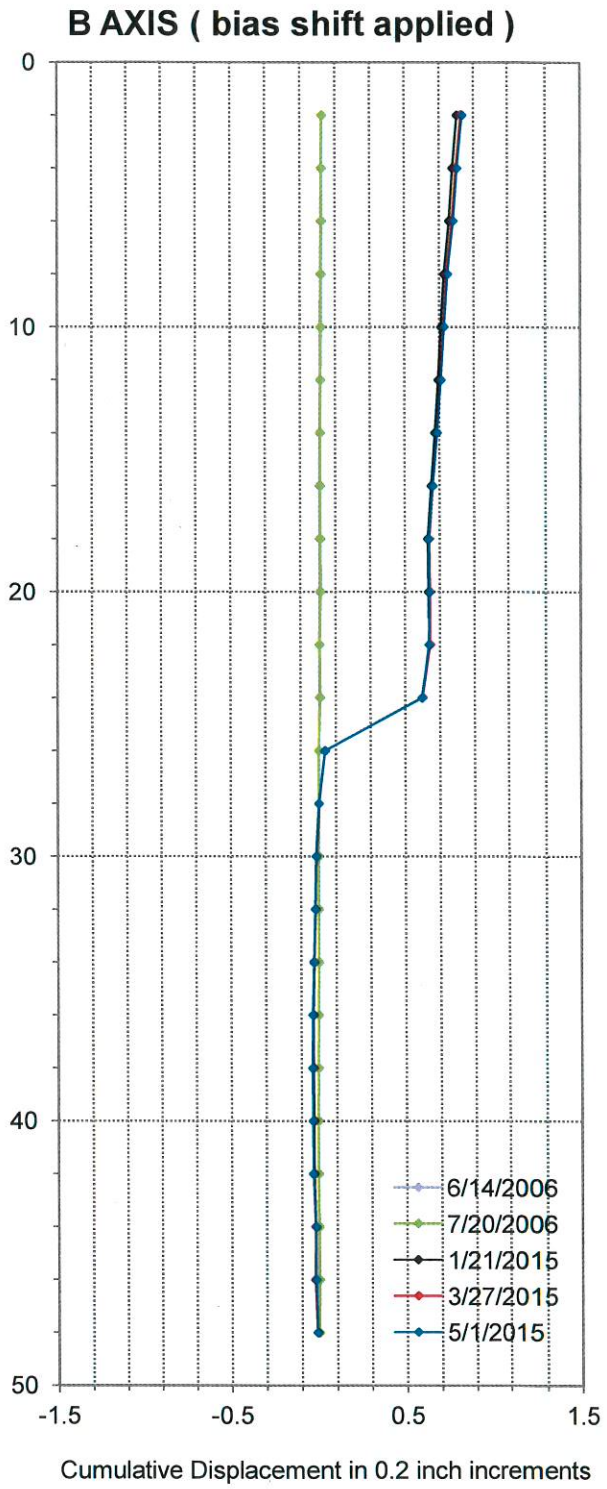
# Appendix C
# SI Data and Plots

# SI-4
# SLIDERANCH
## 04-MRN-1-PM 8.03

| PERIOD | PERIOD LENGTH (DAYS) | PERIOD VECTOR DISP (IN) | PERIOD AVG DISP PER DAY(IN) | PERIOD RATE (IN/YEAR) | LIFETIME VECTOR DISP (IN) | VECTOR DIRECTION ( ° TN ) |
|---|---|---|---|---|---|---|
| 3/27/2015 - 5/1/2015 | 35 | -0.0070 | -0.0002 | -0.07 | 1.9630 | 179.47° |
| 1/21/2015 - 3/27/2015 | 65 | 0.0026 | 0.0000 | 0.01 | 1.9700 | 179.42° |
| 12/10/2014 - 1/21/201 | 42 | 0.0108 | 0.0003 | 0.09 | 1.9674 | 179.37° |
| 9/17/2014 - 12/10/201 | 84 | 0.0042 | 0.0001 | 0.02 | 1.9566 | 179.51° |
| 7/10/2014 - 9/17/2014 | 69 | 0.0021 | 0.0000 | 0.01 | 1.9523 | 179.53° |
| 5/1/2014 - 7/10/2014 | 70 | 0.0003 | 0.0000 | 0.00 | 1.9502 | 179.47° |
| 12/19/2013 - 5/1/2014 | 133 | 0.0101 | 0.0001 | 0.03 | 1.9499 | 179.35° |
| 11/13/2013 - 12/19/20 | 36 | -0.0002 | 0.0000 | 0.00 | 1.9398 | 179.43° |
| 9/3/2013 - 11/13/2013 | 71 | 0.0032 | 0.0000 | 0.02 | 1.9400 | 179.52° |
| 5/21/2013 - 9/3/2013 | 105 | 0.0198 | 0.0002 | 0.07 | 1.9368 | 179.51° |
| 1/23/2013 - 5/21/2013 | 118 | 0.0144 | 0.0001 | 0.04 | 1.9170 | 179.43° |
| 9/7/2012 - 1/23/2013 | 138 | 0.0286 | 0.0002 | 0.08 | 1.9027 | 179.45° |
| 6/27/2012 - 9/7/2012 | 72 | 0.0114 | 0.0002 | 0.06 | 1.8741 | 179.25° |
| 3/12/2012 - 6/27/2012 | 107 | 0.0675 | 0.0006 | 0.23 | 1.8626 | 179.42° |
| 12/16/2011 - 3/12/201 | 87 | 0.0065 | 0.0001 | 0.03 | 1.7951 | 179.48° |
| 10/19/2011 - 12/16/20 | 58 | 0.0028 | 0.0000 | 0.02 | 1.7886 | 179.47° |
| 8/31/2011 - 10/19/201 | 49 | 0.0084 | 0.0002 | 0.06 | 1.7858 | 179.49° |
| 6/30/2011 - 8/31/2011 | 62 | 0.0304 | 0.0005 | 0.18 | 1.7774 | 179.42° |
| 5/19/2011 - 6/30/2011 | 42 | 0.0556 | 0.0013 | 0.48 | 1.7469 | 179.51° |
| 4/26/2011 - 5/19/2011 | 23 | 0.0675 | 0.0029 | 1.07 | 1.6913 | 179.42° |
| 2/24/2011 - 4/26/2011 | 61 | 0.7164 | 0.0117 | 4.29 | 1.6238 | 179.34° |
| 12/16/2010 - 2/24/201 | 70 | 0.1013 | 0.0014 | 0.53 | 0.9073 | 178.81° |
| 9/30/2010 - 12/16/201 | 77 | 0.0043 | 0.0001 | 0.02 | 0.8060 | 179.01° |
| 7/16/2010 - 9/30/2010 | 76 | 0.0148 | 0.0002 | 0.07 | 0.8017 | 178.97° |
| 5/20/2010 - 7/16/2010 | 57 | 0.0214 | 0.0004 | 0.14 | 0.7869 | 178.92° |
| 4/7/2010 - 5/20/2010 | 43 | 0.0904 | 0.0021 | 0.77 | 0.7655 | 178.93° |
| 1/6/2010 - 4/7/2010 | 91 | 0.2377 | 0.0026 | 0.95 | 0.6751 | 178.86° |
| 10/22/2009 - 1/6/2010 | 76 | -0.0002 | 0.0000 | 0.00 | 0.4374 | 178.74° |
| 8/19/2009 - 10/22/200 | 64 | 0.0063 | 0.0001 | 0.04 | 0.4376 | 178.63° |
| 6/18/2009 - 8/19/2009 | 62 | 0.0042 | 0.0001 | 0.02 | 0.4313 | 179.00° |
| 4/22/2009 - 6/18/2009 | 57 | 0.0032 | 0.0001 | 0.02 | 0.4272 | 178.67° |
| 2/24/2009 - 4/22/2009 | 57 | 0.0070 | 0.0001 | 0.04 | 0.4240 | 179.24° |
| 12/2/2008 - 2/24/2009 | 84 | 0.0046 | 0.0001 | 0.02 | 0.4170 | 178.60° |
| 9/17/2008 - 12/2/2008 | 76 | 0.0051 | 0.0001 | 0.02 | 0.4124 | 178.95° |
| 7/16/2008 - 9/17/2008 | 63 | 0.0089 | 0.0001 | 0.05 | 0.4073 | 179.08° |
| 5/23/2008 - 7/16/2008 | 54 | 0.0082 | 0.0002 | 0.06 | 0.3983 | 178.78° |
| 4/10/2008 - 5/23/2008 | 43 | 0.0137 | 0.0003 | 0.12 | 0.3901 | 178.83° |
| 3/13/2008 - 4/10/2008 | 28 | 0.0215 | 0.0008 | 0.28 | 0.3765 | 178.78° |
| 1/16/2008 - 3/13/2008 | 57 | 0.1210 | 0.0021 | 0.78 | 0.3549 | 179.83° |
| 10/29/2007 - 1/16/200 | 79 | 0.0041 | 0.0001 | 0.02 | 0.2339 | 178.96° |
| 8/29/2007 - 10/29/200 | 61 | 0.0041 | 0.0001 | 0.02 | 0.2298 | 181.20° |
| 6/25/2007 - 8/29/2007 | 65 | 0.0126 | 0.0002 | 0.07 | 0.2257 | 179.73° |

# Cumulative Displacement (inches) since 6/7/2006

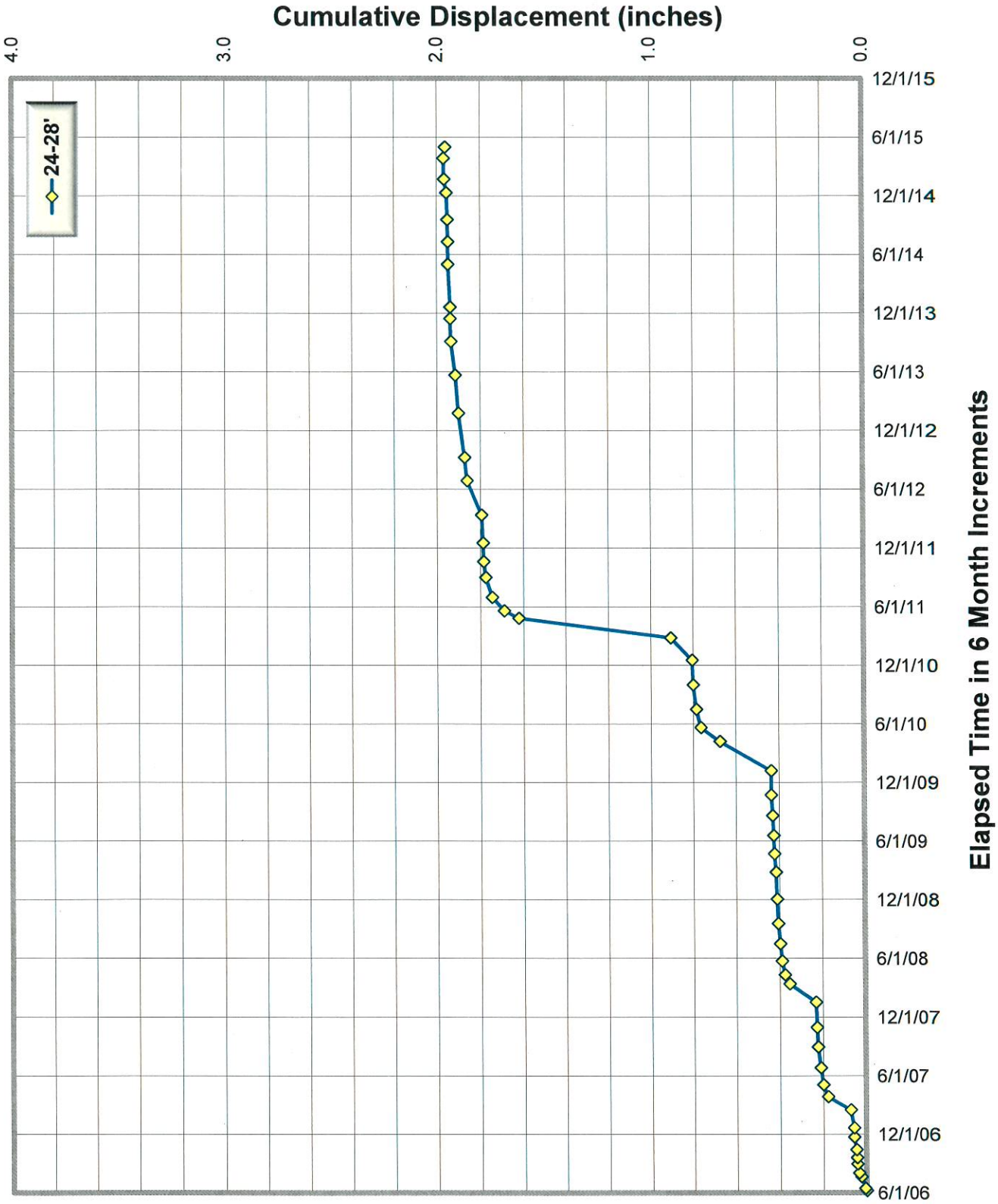



**SI-4**
**SLIDERANCH**
**04-MRN-1-PM 8.03**
May 20, 2015

CA-Department of Transportation
DES-OGDW Orinda Field Station
15 Camino Pablo (925) 254-6504

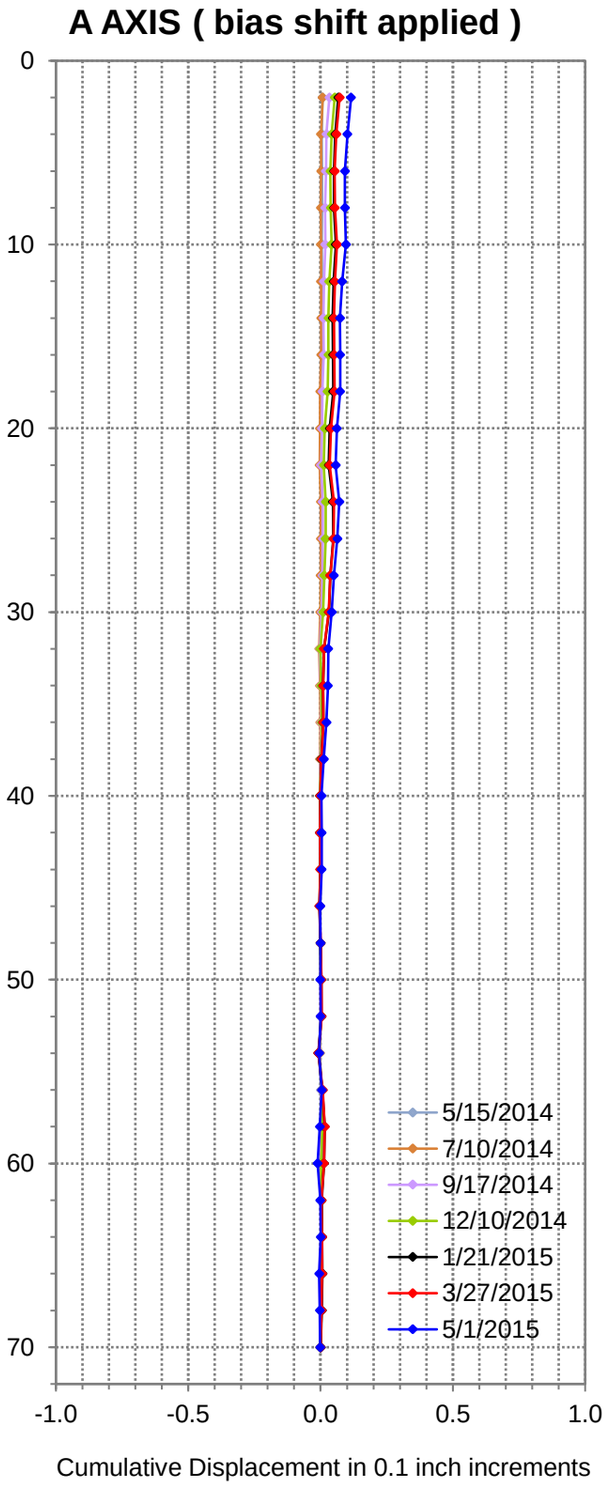
# Cumulative Vector Displacement Over Time

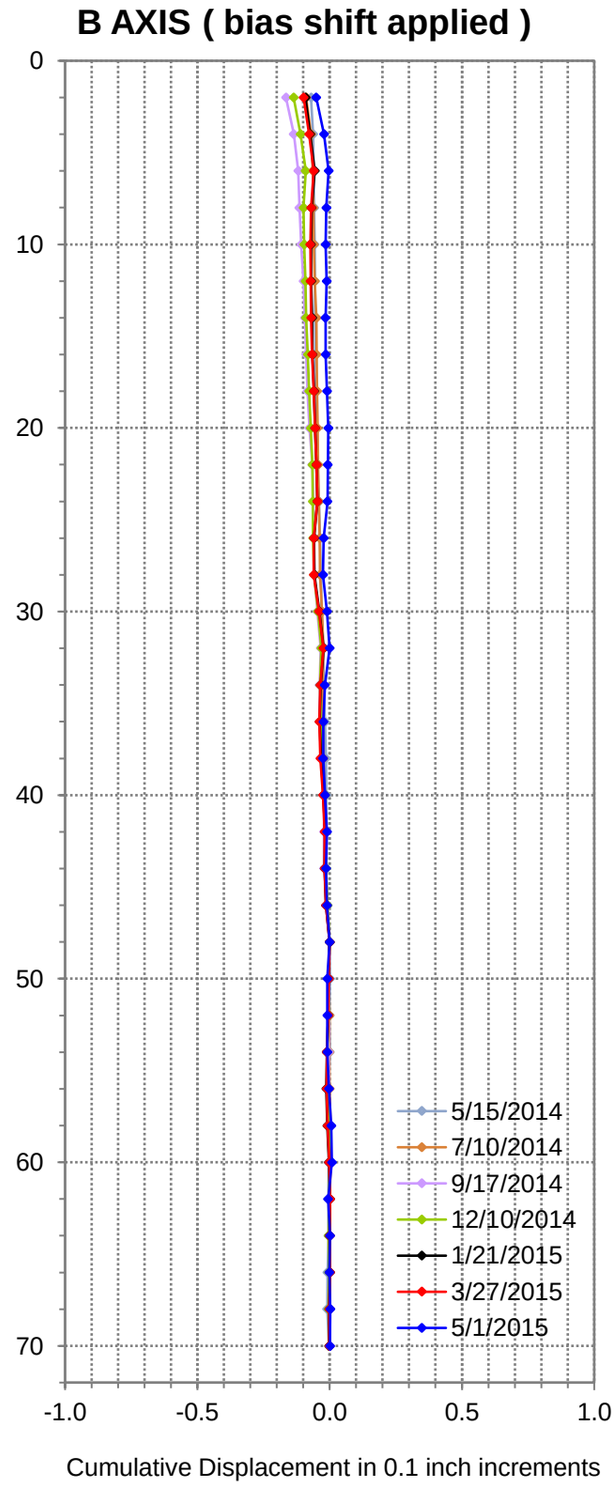



**SI-4**
**SLIDERANCH**
**04-MRN-1-PM 8.03**
May 20, 2015

CA-Department of Transportation
DES-OGDW Orinda Field Station
15 Camino Pablo (925) 254-6504

# Cumulative Displacement (inches) since 5/1/2014



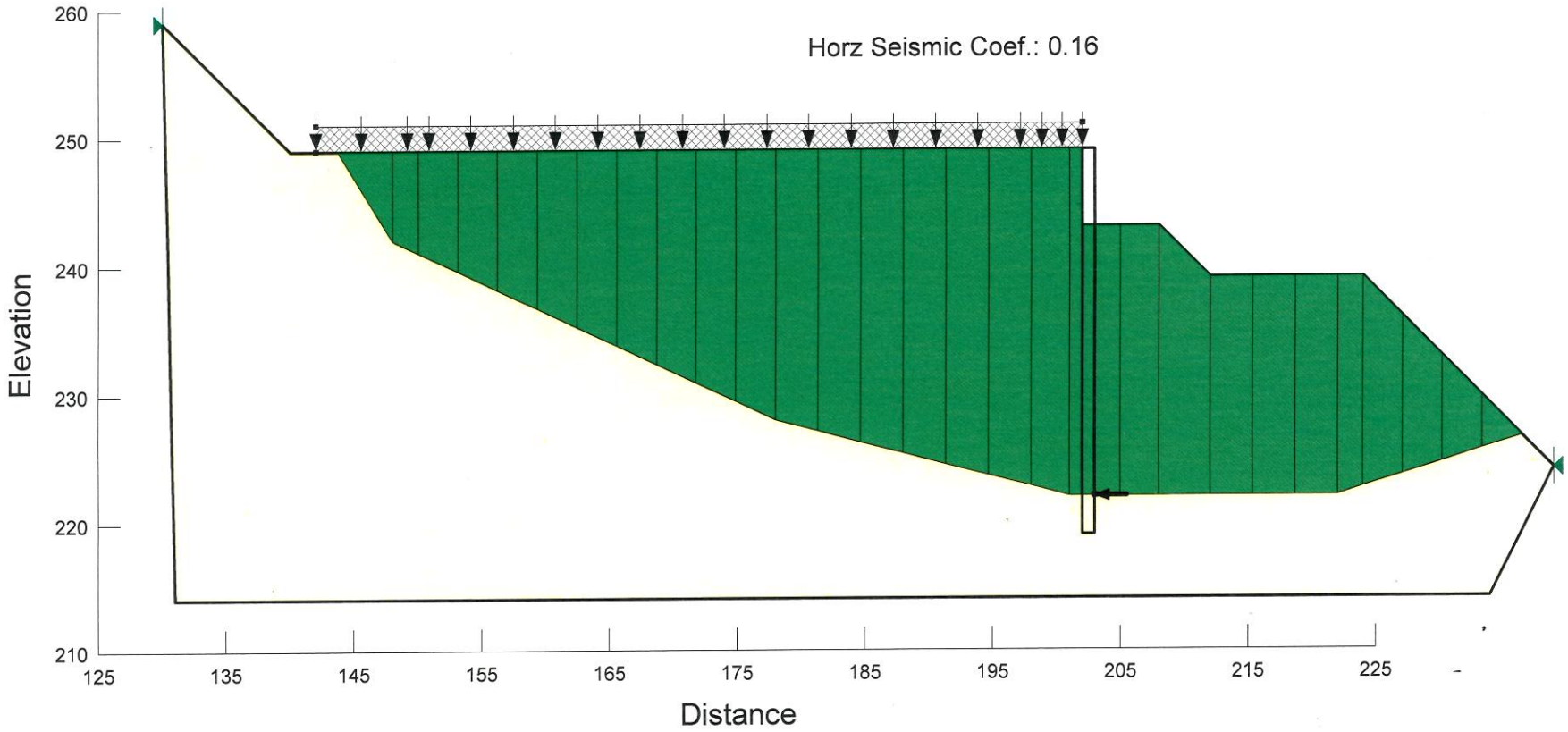

**SI-5**
**SLIDERANCH**
**04-MRN-1-PM 8.0**
October 30, 2015

CA-Department of Transportation
DES-OGDW Orinda Field Station
15 Camino Pablo (925) 254-6504

# Appendix D
# Back Analysis
# Slope Stability

0.953

PARAMETERS
Weight = 130 pcf
Ang Friction= 16 deg
Cohesion = 0 psf
Ru=0.08



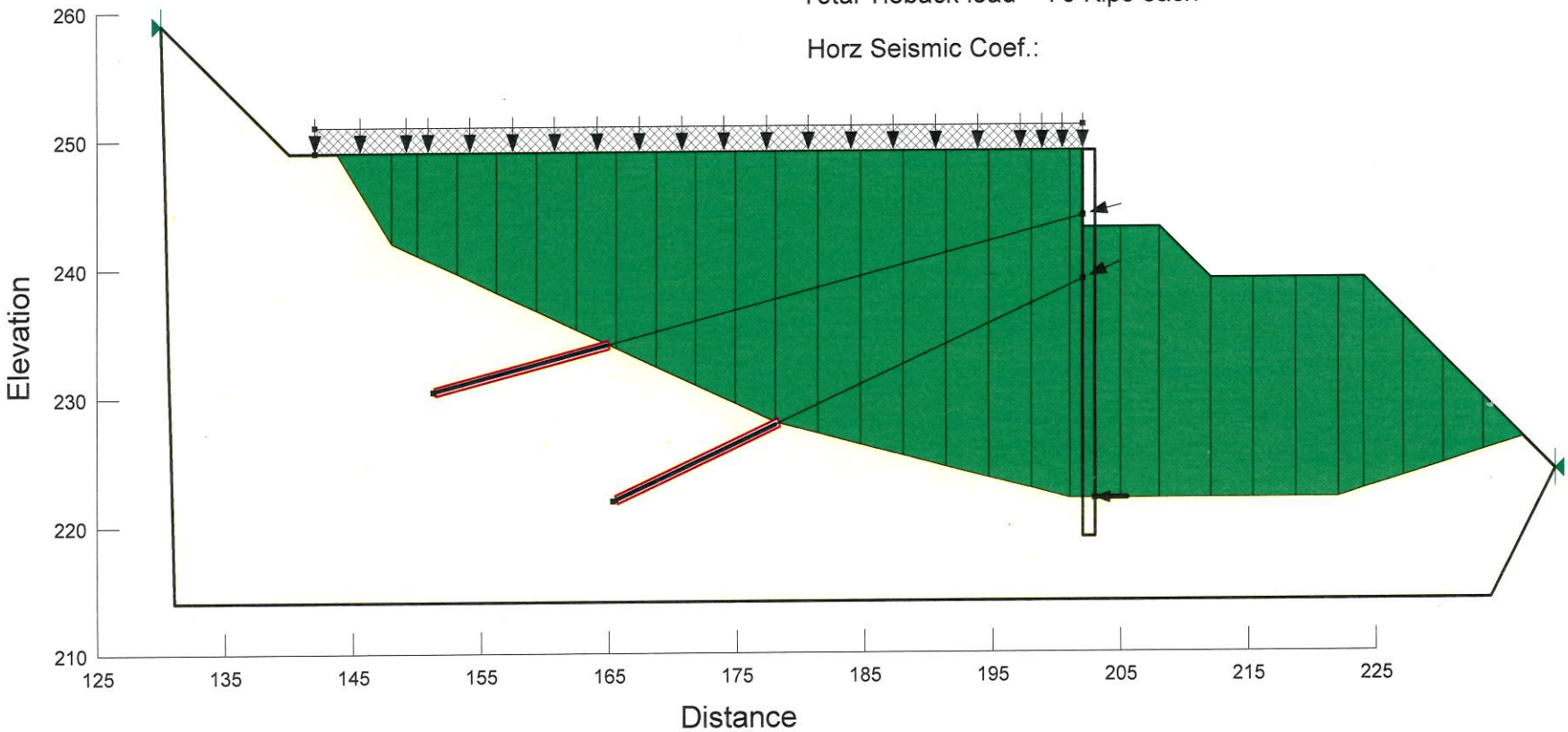

04-MRN-1 PM 8.05

2.137

PARAMETERS
Weight = 130 pcf
Ang Friction= 16 deg
Cohesion = 0 psf
Ru=0.08

Tieback load = 14 Kips/ft each
Tiebacks @ 5 ft
Total Tieback load = 70 Kips each

Horz Seismic Coef.:



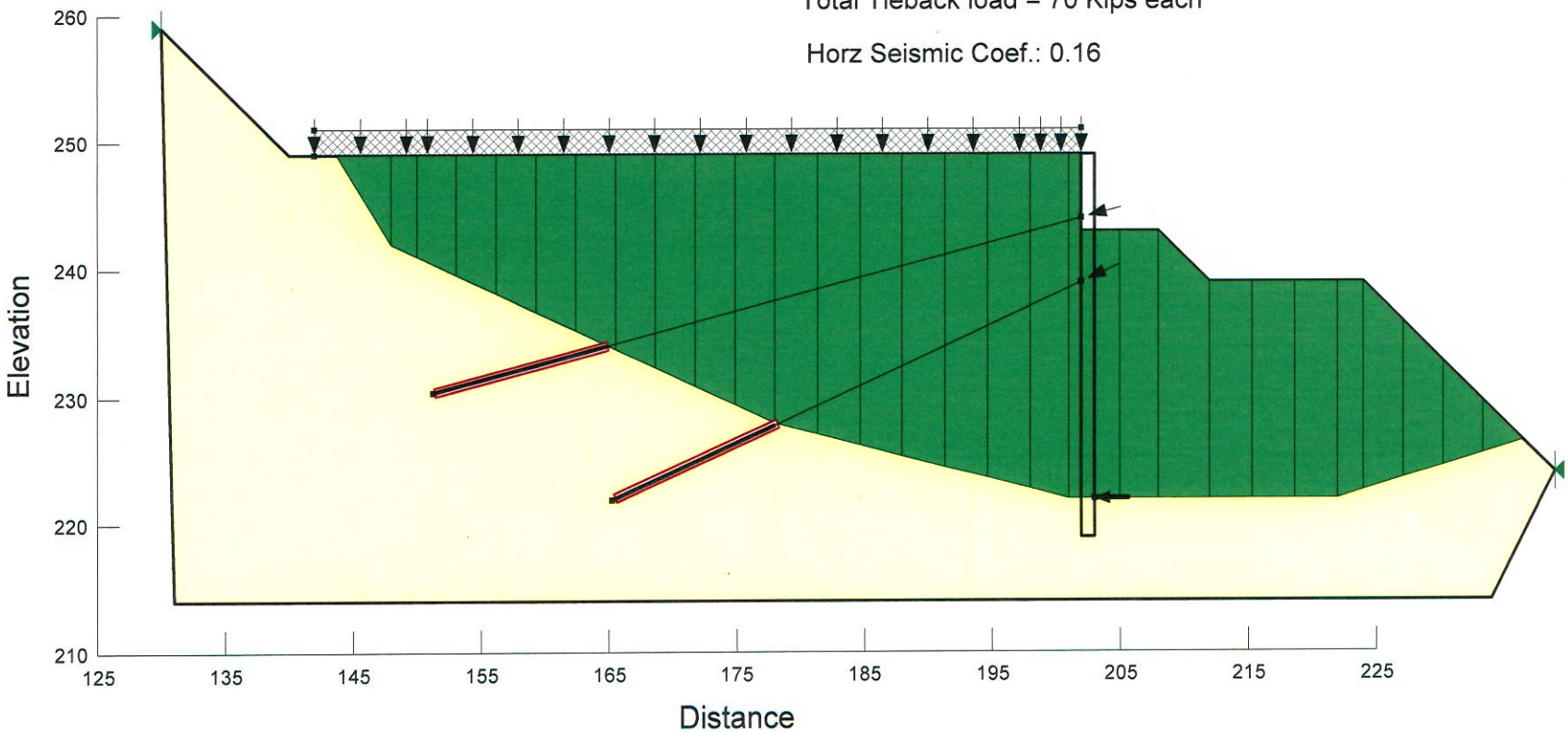

04-MRN-1 PM 8.05

1.153

PARAMETERS
Weight = 130 pcf
Ang Friction= 16 deg
Cohesion = 0 psf
Ru=0.08

Tieback load = 14 Kips/ft each
Tiebacks @ 5 ft
Total Tieback load = 70 Kips each

Horz Seismic Coef.: 0.16




04-MRN-1 PM 8.05

# Appendix E
# Lateral Earth Pressures

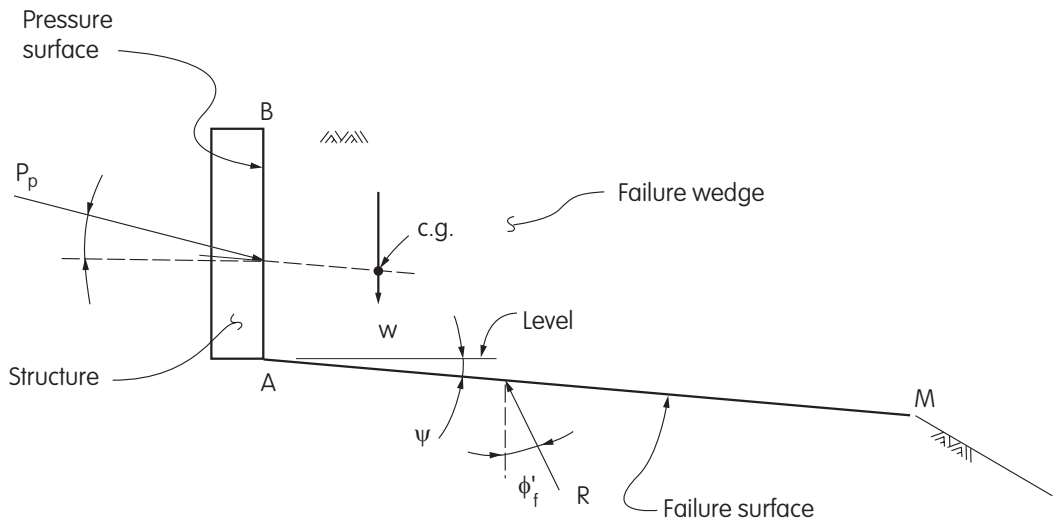



**Figure 5.5.5.5-6 Trial Wedge Method-Passive Pressure, Coulomb's Theory**

Figure 5.5.5.5-6 shows the assumptions used in the determination of the resultant passive pressure for a broken back slope condition applying Coulomb's theory. The pressure surface, AB, moves toward the backfill soil by rotating in a clockwise direction about, A, and may also translate to the right sufficiently to create a passive state of stress in the backfill soil. This movement causes a failure surface to form. It is assumed that this surface is a plane, AM. The wedge of soil, BAM, moves downward along the failure surface and also upward relative to the pressure surface of the structure. This wedge, whose weight is, *W*, is held in equilibrium by the resultant passive pressure, $P_p$ , acting on the surface, AB, and the resultant force, *R*, acting on the failure surface, AM. Since the wedge moves upward along, AB, the force, $P_p$ , acts with an assumed obliquity, *d* , above the normal to oppose this movement. Similarly, the force, *R* , acts with an obliquity, $ø'_f$, to the normal in a direction that opposes movement of the wedge along the failure surface. For any assumed direction of the failure surface, AM, as defined by angle *y* from the horizontal, the directions of, *W*, *R*, and, $P_p$ , are known or assumed, and the magnitude of, $P_p$, can be determined. With the trial wedge method of analysis, the direction of the failure surface, AM, is varied until the determined magnitude of, $P_p$ , is a minimum. The point of application of the resultant passive pressure on the pressure surface is determined by passing a line through the center of gravity (c.g.) of the weight of the failure wedge which is parallel to the failure surface, AM. The point at which this line intersects the pressure surface, AB, is the point of application of the resultant passive pressure.

### 5.5.5.6 Lateral Earth Pressures For Non-Gravity Cantilevered Walls

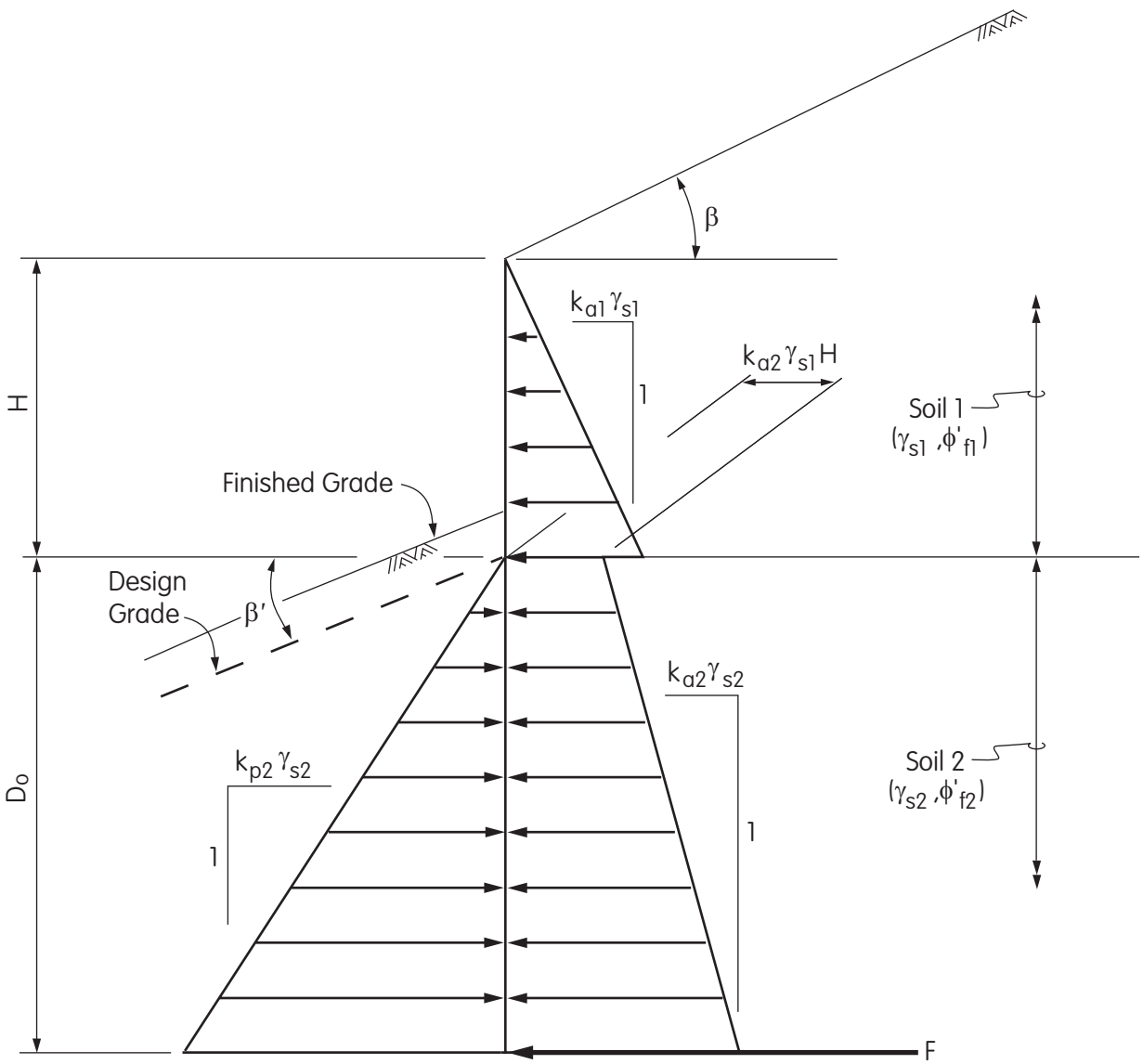
For permanent walls, the simplified lateral earth pressure distributions shown in Figures 5.5.5.6-1 and 5.5.5.6-2 may be used. If walls will support or are supported by cohesive soils for temporary applications, the walls may be designed based on total stress methods of analysis and undrained shear strength parameters. For this latter case, the simplified lateral earth pressure distributions shown in Figures 5.5.5.6-3, and 5.5.5.6-4 may be used with the following restrictions:

- The ratio of total overburden pressure to undrained shear strength, $N_S$ (see Article 5.5.5.7.2), must be <3 at the design grade in front of wall.

- The active lateral earth pressure acting over the wall height, *H*, shall not be less than *0.25* times the effective overburden pressure at any depth, or *0.036* KSF/FT of wall height, which ever is greater.

For temporary walls with vertical elements embedded in granular soil or rock and retaining cohesive soil, Figures 5.5.5.6-1 and 5.5.5.6-2 may be used to determine the lateral earth pressure distributions on the embedded portion of the vertical elements and Figure 5.5.5.6-4 may be used to determine the lateral earth pressure distribution due to the retained cohesive soil.

The lateral earth pressure distributions in Figures 5.5.5.6-1 thru 5.5.5.6-4 shown acting on the embedded portion of vertical wall elements shall be applied to the effective width, *b'*, of discrete vertical wall elements. See Article 5.7.6 for effective widths of discrete vertical wall elements to be used.




Note: The value of β′ is negative for the slope shown.

**Figure 5.5.5.6-1 Simplified Lateral Earth Pressure Distributions for Permanent Non-gravity Cantilevered Walls with Vertical Wall Elements Embedded in Granular Soil and Retaining Granular Soil**

### 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.

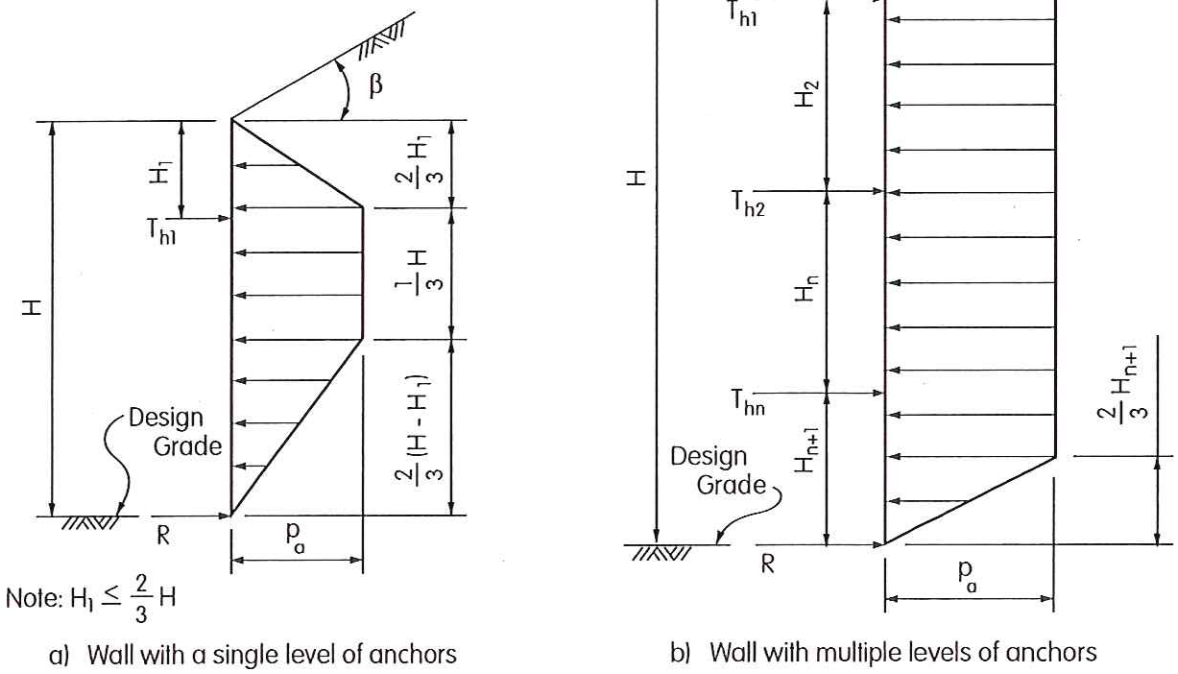
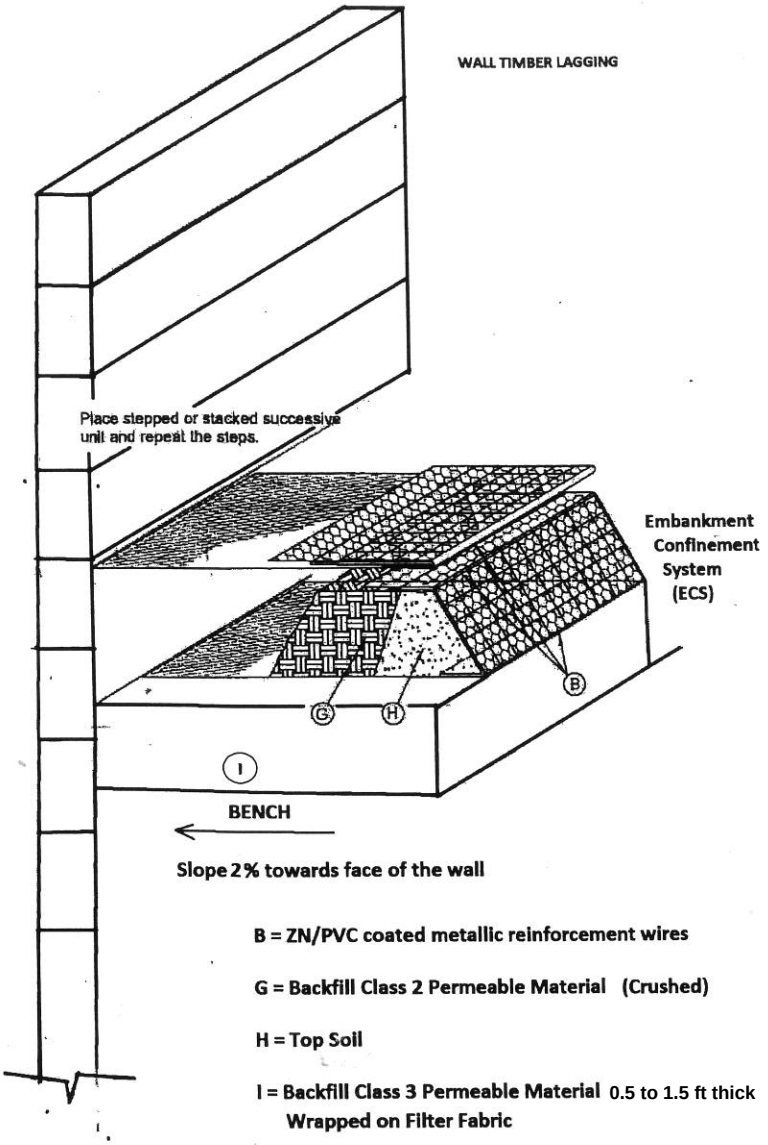



**Figure 5.5.5.7.1-1 Lateral Earth Pressure Distributions for Anchored Walls Constructed from the Top Down in Cohesionless Soils**

# Appendix F
# Embankment Confinement System (ECS)




B = ZN/PVC coated metallic reinforcement wires

G = Backfill Class 2 Permeable Material (Crushed)

H = Top Soil

I = Backfill Class 3 Permeable Material 0.5 to 1.5 ft thick
Wrapped on Filter Fabric

Class 2 and Class 3 Permeable shall be compacted at 90%

| Dist | COUNTY | ROUTE | KILOMETER POST TOTAL PROJECT | SHEET No. | TOTAL SHEETS |
|---|---|---|---|---|---|
| 04 | | | | | |

AS-AWARDED CONTRACT PLANS
THIS STAMP PLACED IN LIEU OF ORIGINAL SIGNATURE

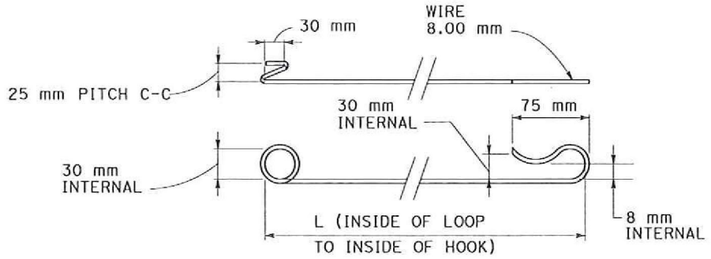
REGISTERED CIVIL ENGINEER DATE

8-4-08
PLANS APPROVAL DATE

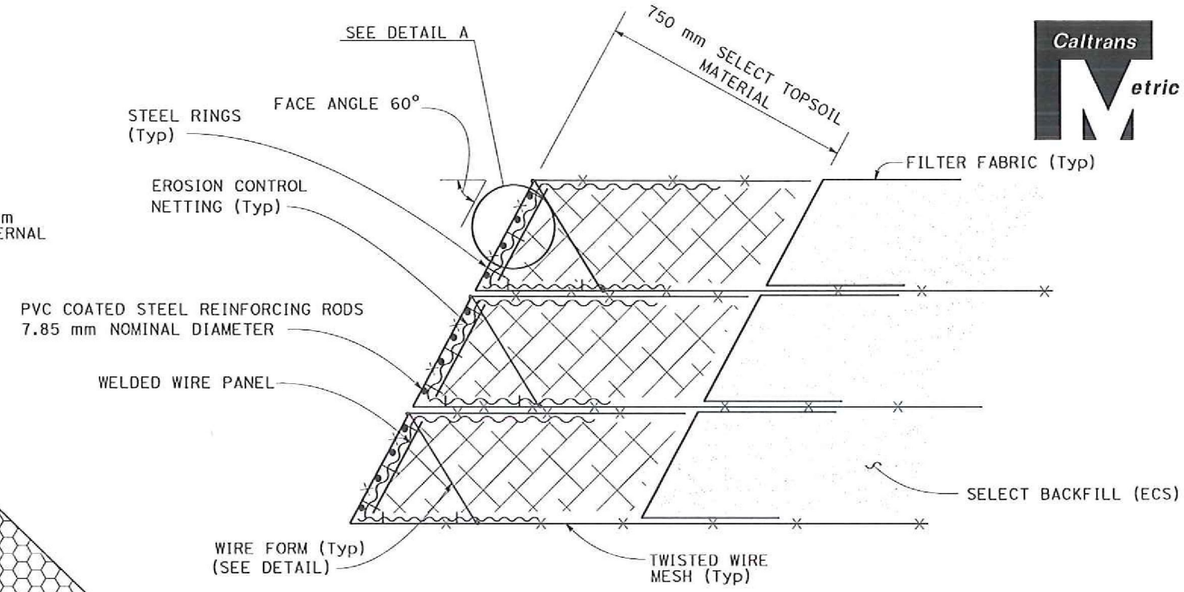
REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

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

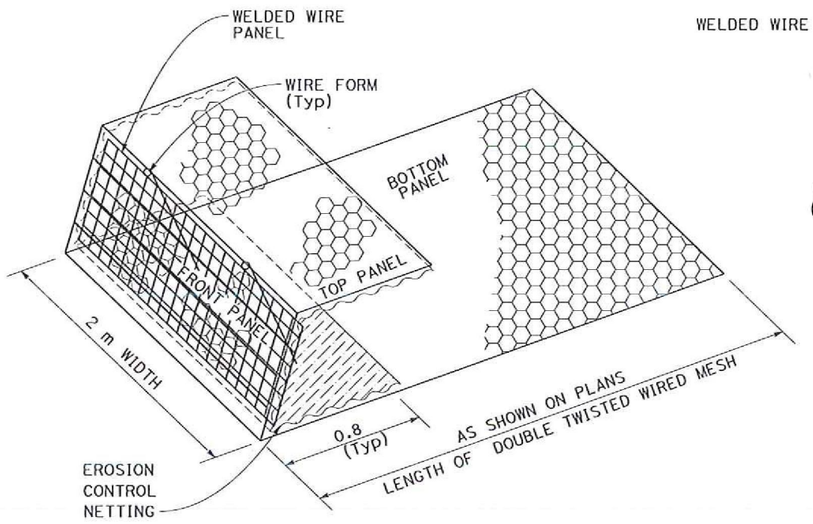
Caltrans Metric

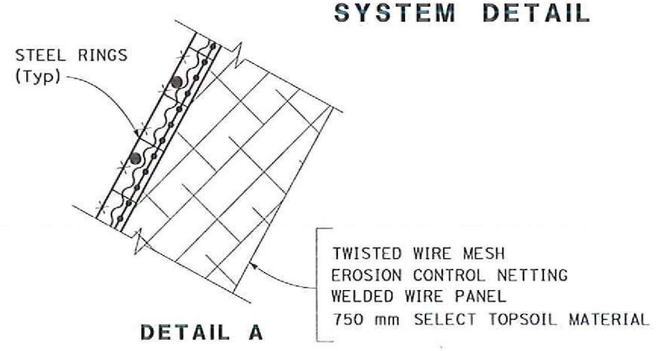



## WIRE FORM DETAIL

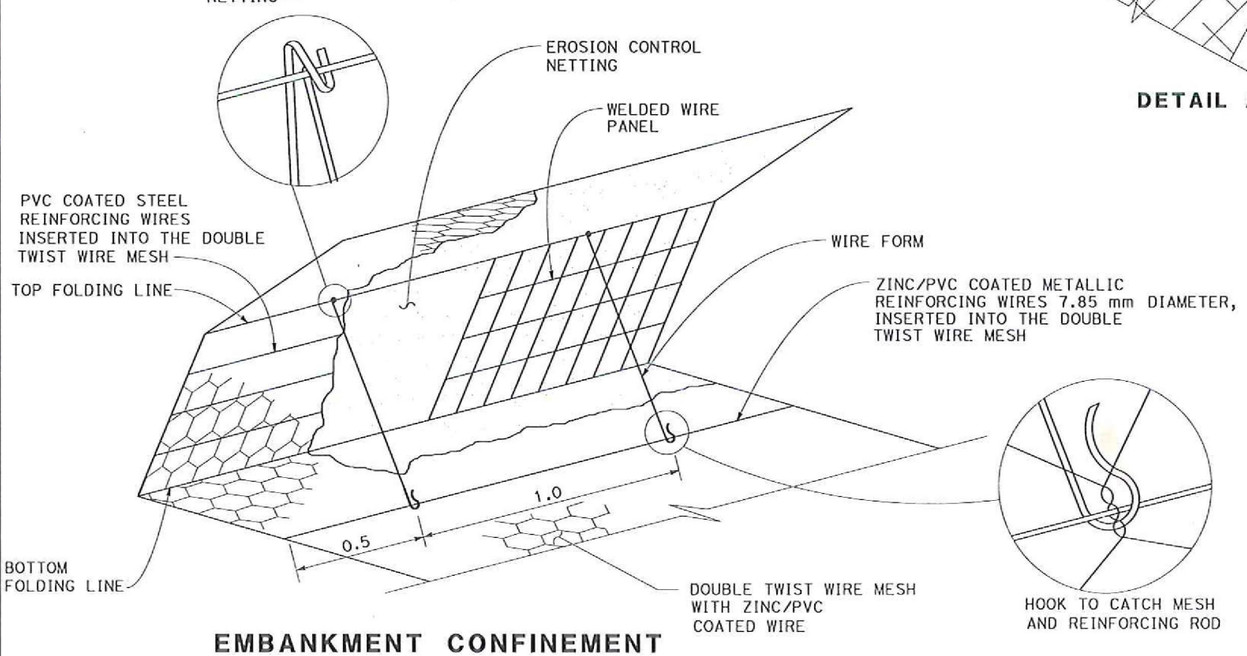



## EMBANKMENT CONFINEMENT SYSTEM DETAIL

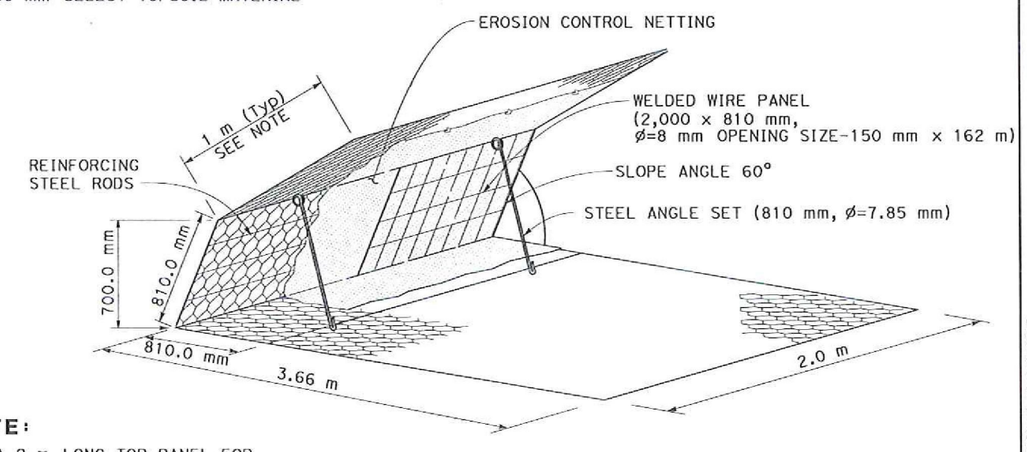



## DETAIL A




## EMBANKMENT CONFINEMENT SYSTEM DETAIL




**NOTE:**
USE A 2 m LONG TOP PANEL FOR LAST ROW OF ECS UNITS ON TOP.

## CONSTRUCTION DETAILS

NO SCALE

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

C-2

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans DESIGN

FUNCTIONAL SUPERVISOR | CALCULATED-DESIGNED BY | CHECKED BY | REVISED BY | DATE REVISED 5/07

LAST REVISION | DATE PLOTTED => 05-FEB-2015

BORDER LAST REVISED 7/1/2007 | RELATIVE BORDER SCALE 0 20 40 60 80 | USERNAME =>

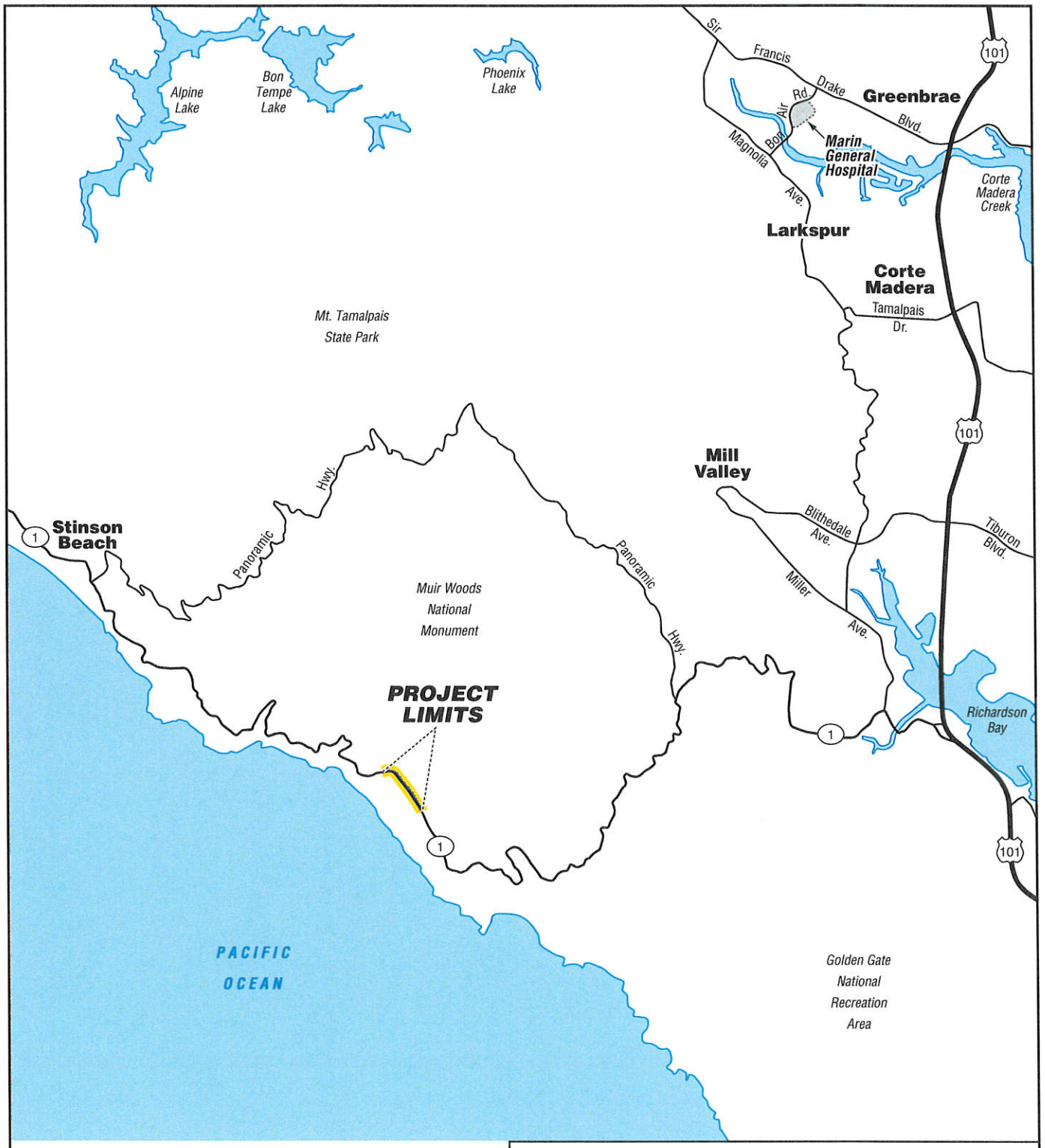
# 2. Soil Aerially Deposited Lead (ADL) and Naturally Occurring Asbestos (NOA) Report

**(April, 2015)**

**Marin County, Highway 1, PM 8.0**

**Project ID-0400025291/ EA-04-2G8904**

**Storm Damage Repair Project**




N

0 1 2

Scale in Miles

GEOCON CONSULTANTS, INC.

6671 BRISA STREET – LIVERMORE, CA 94550
PHONE 925.371.5900 – FAX 925.371.5915

State Route 1 Storm Damage Repair

| | | |
|---|---|---|
| Marin County, California | **VICINITY MAP** | |
| GEOCON Proj. No. E8721-02-27 | | |
| Task Order No. 27 | April 2015 | Figure 1 |

LEGEND:

Boring Location

GEOCON CONSULTANTS, INC.

6671 BRISA STREET, LIVERMORE, CA 94550, PHONE 925 371-5900 - FAX 925 371-5915

| | | |
|---|---|---|
| State Route 1 Storm Damage Repair | | |
| Marin County, California | SITE PLAN | |
| EA No. 04-3S9101 | | |
| GEOCON Proj. No. E8721-02-27 | April 2015 | Figure 2 |

SITE0315

**TABLE 1**
**Boring Coordinates**
**State Route 1 Storm Damage Repair**
**Marin County, California**

| Boring | Latitude | Longitude |
|---|---|---|
| B1 | 37.871662 | -122.590967 |
| B2 | 37.872407 | -122.591710 |
| B3 | 37.872895 | -122.592076 |
| B4 | 37.871743 | -122.590912 |
| B5 | 37.872675 | -122.591730 |
| B6 | 37.873130 | -122.592123 |

**TABLE 2**
**Summary of Lead and pH Results - Soil**
**State Route 1 Storm Damage Repair**
**Marin County, California**

| Sample ID | Sample Depth (feet) | Total Lead (mg/kg) | pH |
|---|---|---|---|
| B1-0 | 0 to 0.5 | 10 | --- |
| B1-1 | 1 to 1.5 | 6.1 | --- |
| B1-2.5 | 2.5 to 3 | 8.6 | 7.2 |
| B1-4.5 | 4.5 to 5 | 8.8 | --- |
| B1-9.5 | 9.5 to 10 | 5.5 | --- |
| B1-19.5 | 19.5 to 20 | 7.9 | --- |
| B2-0 | 0 to 0.5 | 12 | --- |
| B2-1 | 1 to 1.5 | 7.7 | 6.5 |
| B2-2.5 | 2.5 to 3 | 5.2 | --- |
| B2-4.5 | 4.5 to 5 | 26 | --- |
| B2-9.5 | 9.5 to 10 | 1.5 | --- |
| B2-19.5 | 19.5 to 20 | <5.0 | --- |
| B2-39.5 | 39.5 to 40 | 1.1 | --- |
| B3-0 | 0 to 0.5 | 16 | --- |
| B3-1 | 1 to 1.5 | 8.5 | --- |
| B3-2.5 | 2.5 to 3 | 7.4 | --- |
| B3-4.5 | 4.5 to 5 | 3.6 | 6.8 |
| B3-9.5 | 9.5 to 10 | 9.0 | --- |
| B3-18 | 18 to 18.5 | 3.1 | --- |
| B4-0 | 0 to 0.5 | 6.1 | --- |
| B4-1.5 | 1 to 1.5 | 8.6 | 6.2 |
| B5-0 | 0 to 0.5 | 12 | --- |
| B5-1.5 | 1 to 1.5 | 12 | 6.2 |
| B6-0 | 0 to 0.5 | 10 | --- |
| B6-1.5 | 1 to 1.5 | 13 | 5.9 |
| RB | | <0.0050 (mg/l) | |
| Hazardous Waste Criteria | | | |
| | TTLC (mg/kg) | 1,000 | --- |

**Notes:**
mg/kg = Milligrams per kilogram
mg/l = Milligrams per liter
TTLC = Total Threshold Limit Concentration

**TABLE 5**
**Summary of NOA Results**
**State Route 1 Storm Damage Repair**
**Marin County, California**

| Sample ID | Sample Depth (feet) | Asbestos Content |
|---|---|---|
| B1-0 | 0 to 0.5 | ND |
| B1-1 | 1 to 1.5 | ND |
| B1-2.5 | 2.5 to 3 | ND |
| B1-4.5 | 4.5 to 5 | ND |
| B1-9.5 | 9.5 to 10 | ND |
| B1-19.5 | 19.5 to 20 | ND |
| B3-0 | 0 to 0.5 | ND |
| B3-1 | 1 to 1.5 | ND |
| B3-2.5 | 2.5 to 3 | ND |
| B3-4.5 | 4.5 to 5 | ND |
| B3-9.5 | 9.5 to 10 | ND |
| B3-18 | 18 to 18.5 | ND |
| B2-0 | 0 to 0.5 | ND |
| B2-1 | 1 to 1.5 | ND |
| B2-2.5 | 2.5 to 3 | ND |
| B2-4.5 | 4.5 to 5 | ND |
| B2-9.5 | 9.5 to 10 | ND |
| B2-19.5 | 19.5 to 20 | ND |
| B2-39.5 | 39.5 to 40 | ND |
| B4-0 | 0 to 0.5 | ND |
| B4-1.5 | 1.5 to 2 | ND |
| B5-0 | 0 to 0.5 | ND |
| B5-1.5 | 1.5 to 2 | ND |
| B6-0 | 0 to 0.5 | ND |
| B6-1.5 | 1.5 to 2 | ND |

ND = None detected at 0.25% target analytical sensitivity.