FOUNDATION REPORT

US 50, Silva Valley Parkway Overcrossing
El Dorado County, California
Bridge No. 25-0127
03-ED-50
PM R1.8
EA 03-1E2901

Prepared for:

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April 2012

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File No. 556.2
April 20, 2012

Mrs. Julie Passalacqua
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Subject: **FOUNDATION REPORT**
US 50, Silva Valley Parkway Overcrossing
Bridge No. 25-0127
03-ED-50; PM R1.8; EA 03-1E2901

Dear Mrs. Passalacqua,

In accordance with our April 7, 2010 agreement, Blackburn Consulting (BCI) prepared this Foundation Report for the Overcrossing planned for the US50 / Silva Valley Parkway Interchange project.

This report contains our subsurface findings, conclusions and recommendations for foundation design. We also submitted our Preliminary Foundation Report (PFR) for review on August 18, 2010 and our Draft Foundation Report on November 3, 2010.

Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Foundation Report for the new overcrossing (OC) planned for the US 50/Silva Valley Parkway Interchange project in El Dorado Hills, El Dorado County, California.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of the subsurface conditions, and to recommend geotechnical design and construction criteria for the proposed bridge. Do not use or rely upon this report for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Reviewed preliminary bridge design plans provided by Mark Thomas and Company, Inc. (MTCo)
- Discussed the project design needs with MTCo
- Reviewed geologic and seismic maps pertaining to the site
- Drilled and sampled three (3) diamond core borings to a maximum depth of 51 feet below existing grade at the abutments and bent
- Reviewed the existing highway cut slopes in rock below the abutment locations
- Performed laboratory testing on soil and rock samples retrieved from the borings
- Performed engineering and seismic analysis to provide recommendations for structure foundations and approach

This Foundation Report supersedes the Preliminary Foundation Report by BCI for the Silva Valley Parkway OC dated August 18, 2010 and the Draft Foundation Report dated November 3, 2012.

2 PROJECT DESCRIPTION

2.1 Project Location and Site Description

The project is located in El Dorado County, California, along US 50, Post Mile (PM) R1.8, approximately 800 feet east of the existing Clarksville Undercrossing (UC, Br. No. 25-0072 at the existing Silva Valley Parkway). Figure 1 (Vicinity Map), in Appendix A, shows the approximate project location.

At the OC location, US 50 consists of two lanes in each direction with an HOV lane currently under construction at the median. The OC site crosses a north-northwest trending ridge with a through-cut for US 50. The existing cut slopes are at a gradient of approximately 1.5H:1V (horizontal to vertical) and the maximum slope heights are approximately 34 feet on the north side (westbound shoulder) and 27 feet on the south side (eastbound shoulder). The slopes are cut into variably weathered and fractured rock.

There are no existing structures at the proposed bridge location. The closest existing bridge structure is the Clarksville UC (Br. No. 25-0072) at (old) Silva Valley Parkway. The existing Clarksville UC consists of two parallel bridges constructed in 1965. Each bridge is an approximately 40-foot-wide by 110-foot-long, three-span structure. The substructure of each bridge consists of open-style abutments supported on short H-piles and two-column bents supported on spread footings. The bridges were widened to the median (infill structure) in 2009/2010 for the HOV lane project with similar pile/footing support.

The vertical datum used for this project (per MTCo) is National Geodetic Vertical Datum 1929 based on HPGN D CA 03 DL having an elevation of 693.55 feet, and USGS BM T 127 (PID JS0692) having an elevation of 673.08 feet.

2.2 Proposed Structure

The project consists of a new overcrossing structure for Silva Valley Parkway. According to MTCo, this structure will be a two span, post-tensioned, concrete box girder bridge, approximately 279 feet long and 105 feet wide. Roadway elevation at the OC abutments/approach will be several feet below existing grade (minor cut will occur – no significant approach fill at the abutments). The existing cut slopes in front of the abutment locations will be cut back (in two phases at the north abutment) for other interchange improvements, match the existing slope at a gradient of approximately 1.5H:1V (horizontal to vertical), and will be finished with slope paving.

The General Plan and Foundation Plan prepared by MTCo (see Appendix B) shows the overcrossing structure supported with a four-column bent at mid-span. Foundations will consist of spread footings, 9 to 11.5 feet wide, at Abutments 1 and 3, and four, 13.5x13.5 feet, spread footings at Bent 2 that are spaced at 24 feet center-to-center.

A retaining/wing wall, Standard Type 1, is located on the east side of Abutment 3. The wall is approximately 26 feet long and varies in design height from 14 to 18 feet. Foundations step up behind the abutment from elevation 732.5 to 737.0 feet.

2.3 Existing Facilities

There are no existing facilities at the overcrossing location except for overhead electric located just south of Abutment No. 1 (south side of US 50) and a possible cemetery located west of Abutment No. 3.

3 SUBSURFACE INVESTIGATION

To characterize the subsurface conditions and obtain samples for laboratory testing, BCI retained PC Exploration to drill and sample three borings at the site in July 2010. PC used a CME 75 truck-mounted rig to drill the borings with 8-inch O.D. hollow-stem augers to relatively competent bedrock, and then HQ, wireline, diamond core equipment to complete the borings in rock. Core diameter is approximately 3.8 inches. The maximum depth of the borings is ±51.0 feet below the ground surface (bgs).

We obtained relatively undisturbed samples in soil and decomposed rock using Modified California Samplers (equipped with 2.4-inch I.D. brass liners). Samplers were driven into the ground with a 140-pound, automatic hammer falling 30 inches. We obtained continuous rock samples from the boring and placed them in labeled core boxes.

BCI's geologist logged the borings consistent with the Unified Soil Classification System (USCS), and noted the degree of weathering, fracture density, hardness percent recovery and Rock Quality Designation (RQD) for the recovered rock cores. BCI also made ground water observations in the augered portion of borings during drilling operations. At the completion of fieldwork, the borings were backfilled with cement-grout.

BCI planned the general location and depth of the borings based on the proposed improvements and existing site/rock conditions. We show investigation points on the Log of Test Borings (LOTB) in Appendix B. The LOTB for this study provides soil and rock descriptions and an explanation of the descriptive terms used to log the soil samples and rock cores.

4 LABORATORY TESTING

In addition to field blow counts (in the upper 2 feet) and Rock Quality Designation (RQD) values, we completed unconfined compression tests of rock (ASTM D 2938) for strength parameters.

We attach laboratory test results in Appendix C.

5 SITE GEOLOGY

5.1 Topography

Within the overcrossing area, US 50 passes through a ridge of moderately sloping ground ranging in elevation from 720 to 757 feet. The existing cut slopes are at a gradient of approximately 1.5H:1V and the maximum slope heights are approximately 34 feet on the north side (westbound shoulder) and 27 feet on the south side (eastbound shoulder). The cuts expose variably weathered and fractured rock. Natural ground slopes away from the ridge at gradients of approximately 5H:1V to 10H:1V.

At Bent 2, located in the US 50 median, the existing ground surface is cut-to-grade (over 25 feet) and relatively flat with elevation ranging from 720 to 723 feet (sloping down to the west) across the bent location.

5.2 Regional Geology

The site is located within the foothills of the Sierra Nevada Geomorphic Province of California. The Sierra Nevada has a general northwest topographic/structural trend and is approximately 430 miles long and 40 to 80 miles wide. The mountain ranges of the Sierra Nevada began to develop roughly 120 to 130 million years ago when sediments as thick as 30,000 feet along with volcanic rocks buckled and warped resulting in a series of low mountain ranges. The roots of these mountain ranges were intruded by granitic rock.

The Sierra Nevada was tilted upward (down to the west) along faulting at the eastern edge. In the higher elevations, much of the younger sedimentary material and older metamorphic rock is eroded and now exposes the underlying granitic rock. Older rocks that remain are metamorphic and are exposed in the foothills of the Sierra Nevada.

Mesozoic-age metavolcanic and metasedimentary rocks underlie most of El Dorado County. Northwest trending foliation and northwest trending faults and fault zones that mark the boundaries of major rock types dominate the metamorphic rock structure.

5.3 Site Geology and Faulting

Published geologic mapping by Wagner[1] and Busch[2] shows Jurassic-age metavolcanic rock at the project site. Our site review and borings confirm the presence of shallow, metavolcanic rock. We show local site geology on Figure 2 (Geologic Map).

Rock structure at the OC location is similar to the surrounding area and has a predominant foliation with a strike of north, 30º-50º west, and a steep dip of 76º-88º to the north (nearly perpendicular to both cut slope faces); along which most fractures occur. We recorded other fractures/discontinuities exposed in the cut slopes, but most are random and discontinuous. Fractures appear generally closed (tight) and rough. Table 1 lists pervasive discontinuities that we recorded.

Table 1 – Pervasive Rock Discontinuities

General Location and Existing Condition	Strike and Dip (degrees)*
Silva Overcrossing, South Abutment, A3R Line, Station 103+00 to 109+00 Existing north-northwest facing slope at 1.5h:1v gradient, maximum height up to approximately 27 feet	N19W, 88S fol N40W, 82N fol N63E, 70S fr N86W, 70S fr N30W, 85N fol N35W, 82N fol N48W, 84N fol

[1] Wagner, D.L. et al, “Geologic Map of the Sacramento Quadrangle, California”, California Geological Survey, Map No. 1A, 1981, revised 1987.
[2] Busch, “Generalized Geologic Map of El Dorado County, California”, June, 2001, California Geological Survey, OFR 2000-03.

General Location and Existing Condition	Strike and Dip (degrees)*
Silva Overcrossing, North Abutment, A3L Line, Station 102+00 to 109+00 Existing south-southeast facing slope at 1.5h:1v gradient, maximum height up to approximately 34 feet	N40W, 83N fol N35W, 76N fol N50E, 84S fr N60E, 16S fr N38W, 84N fol N63E, 70S fr N65W, 25S fr

*fr = fracture, fol = foliation, sh = shear

We did not observe indications of slope instability at the existing cut slopes or natural slopes in the area. The existing cut slopes appear grossly stable at gradients of 1.5H:1V and steeper, and we did not observe significant rockfall, spalling, slab or wedge failures on the slopes. We did not observe groundwater seepage in the OC area or from the adjacent cut slopes.

The West Bear Mountains Fault is located approximately 4,000 feet west of the site (near Latrobe Road) with a short splay mapped approximately 2,000 feet west of the site. The East Bear Mountains Fault (or Rescue section) is located approximately 7 miles east of the site. Faults are not mapped through or adjacent to the OC site and we observed no indication of active faulting in the area.

We did not observe significant occurrence of ultramafic rock where naturally occurring asbestos minerals (NOA) are likely to occur. Published mapping and site review does not indicate that the project is within an ultramafic rock area; however, ultramafic rock and faulting are mapped nearby and naturally occurring asbestos minerals could potentially occur in the area. Geologic mapping by Churchill[3] shows an "area more likely to contain naturally occurring asbestos" about one mile north of the Latrobe Road Undercrossing and east of Bass Lake Road. The mapping shows the site to be within an area "that probably does not contain asbestos."

Mapping by Bruyn[4] shows the OC site on the eastern border of a "Quarter Mile Buffer for More Likely to Contain Asbestos or Fault Line." Churchill discusses the possibility of serpentine occurring in faults or within fault zones, which may contain chrysotile or tremolite/actinolite asbestos.

[3] Churchill, et al., 2000, "Areas More Likely to Contain Natural Occurrences of Asbestos in Western El Dorado County, California", California Geological Survey, OFR 2000-02
[4] Bruyn, 2005, "Asbestos Review Areas, Western Slope, County of El Dorado, State of California", El Dorado County

6 SUBSURFACE CONDITIONS

6.1 Subsurface Soil and Rock Conditions

In general, hard rock occurs at relatively shallow depths throughout the site. The cut slopes at the OC location expose hard rock at depths of 5 to 15 feet with intensely to moderately weathered and fractured rock above that.

At the abutments, our borings (located behind the top of the cut slopes) encountered 1 to 2 feet of silty sand and clay with gravel over intensely to moderately weathered rock that becomes less weathered (moderate to slight) at a depth of 12 to 18 feet. Core recovery was generally above 50 percent to depths of 16 feet and 100 percent at greater depths. Rock Quality Designation (RQD) indicates poor to fair quality rock (RQD of 0 to 50%) to depths of 15 to 20 feet and fair to excellent quality rock (RQD of 50% to 100%) below those depths.

The median bent location is within a highway through-cut and is cut down at least 25 feet from original grade. Our boring at the bent location encountered hard, slightly weathered, metavolcanic rock below approximately 2.5 feet of poorly graded gravel (roadway fill). Core recovery at this location was 100 percent and RQD indicates excellent quality rock (RQD of 96% and greater).

Refer to the attached LOTB in Appendix B for more specific soil and rock descriptions.

6.2 Groundwater

We did not encounter free groundwater to elevation 743 feet within the augered portions of the borings drilled in July 2010. The existing cut slopes did not have groundwater seepage at the time of our field exploration. We did not evaluate groundwater occurrence in the diamond-cored borings due to the presence of drilling fluids.

Although we did not observe groundwater seepage at the surface or within the augered portions of our borings, we expect that shallow groundwater and seepage can occur along the soil/rock interface during the winter months or extended periods of rainfall. Locally, seepage can also occur along zones of fractured or less weathered rock and daylight at the ground surface, within excavations, or onto cut-slopes.

7 SCOUR EVALUATION

The site is not located adjacent to any waterway; therefore, scour is not a consideration for this project.

8 CORROSION EVALUATION

Hard, metavolcanic rock is present at abutment and bent foundation elevations. The rock is not considered corrosive to structural elements.

9 SEISMIC RECOMMENDATIONS

9.1 Fault Rupture

The site does not lie within or adjacent to an Alquist–Priolo Earthquake Fault Zone for fault rupture hazard (Bryant and Hart, 2007)[5], and no known active faults are mapped with the project area. Busch (2001) shows the main trace of the West Bear Mountains Fault crossing US 50 about 4,000 feet west of the OC and a north-south trending splay associated with this fault crossing US 50 about 2,000 feet west of the OC. Jennings (1994)[6] shows the West Bear Mountains Fault as Pre-Quaternary in age. The Caltrans Deterministic PGA Map (September 2007) does not show this fault as an active seismic source and shows no active faults in the project area. The closest fault considered in ground motion analysis is the East Bear Mountains Fault (or Rescue section, Caltrans Fault Identification No. 83) located approximately 7 miles east of the site.

We consider the potential for fault rupture at the site to be low.

9.2 Ground Motion

Based on Caltrans ARS Online (V1.0.4) and other mapping, the closest recognized Late Quaternary or younger fault is the Bear Mountains Fault Zone (Rescue Fault section, Caltrans Fault Identification No. 83, Maximum Moment Magnitude [MMax] = 6.5) located ±7 miles east of the site. Figure 3, Seismic Hazard Map in Appendix A, shows the approximate fault locations.

We used the Caltrans ARS Online (web-based tool) to calculate both deterministic and probabilistic acceleration response spectra for the site based on criteria provided in Appendix B of the Caltrans Seismic Design Criteria (Revision Date:9/11/09).

The deterministic spectrum is determined as the average of median response spectra calculated using ground motion prediction equations developed under the "Next Generation Attenuation" (NGA) project. These equations are applied to all faults considered to be active in the last 700,000 years (late-Quaternary age) that are capable of producing a moment magnitude earthquake of 6.0 or greater.

The probabilistic spectrum is from the USGS (2008) National Hazard Map for 5% probability of exceedance in 50 years (including a deaggregation for applicable fault distance). Caltrans bases design spectrum on the larger of the deterministic and probabilistic spectral values. Both the deterministic and probabilistic spectra account for soil effects through incorporation of the parameter Vs30, the average shear wave velocity in the upper 30 meters of the soil profile. For the project site, we assume a Site Class B/C with Vs30 equal to 760 meters per second (approximately 2,500 feet per second) based on the mapped ground conditions (underlain by shallow metamorphic rock).

[5] Fault Rupture Hazard Zones in California, Special Publication 42, Interim Revision; California Geological Survey
[6] Fault Activity Map of California and Adjacent Areas, Geologic Map No. 6, California Division of Mines and Geology

In general, the minimum deterministic spectra controls at shorter site periods and the probabilistic spectra controls at longer periods (above about 0.7 seconds). The peak ground acceleration (PGA) at the site is approximately 0.20g based on Caltrans ARS Online and minimum deterministic levels of ground acceleration. Design spectrum is based on the upper envelope spectral values of the combined minimum deterministic and probabilistic response spectra across the period spectrum from 0 to 5 seconds. A MMax of 6.5 with a PGA of 0.20g is applicable to the site. We present data points for site spectra in the Table 2 below and graphed site spectra in Figure 4 (Appendix A).

Table 2 - Caltrans ARS Online Envelope* Spectrum Data

Period	SA	Period	SA	Period	SA	Period	SA
0	0.197	**0.085**	0.376	**0.35**	0.333	**1.4**	0.091
0.01	0.197	**0.09**	0.389	**0.36**	0.327	**1.5**	0.086
0.02	0.201	**0.095**	0.401	**0.38**	0.315	**1.6**	0.082
0.022	0.204	**0.1**	0.414	**0.4**	0.303	**1.7**	0.078
0.025	0.208	**0.11**	0.43	**0.42**	0.291	**1.8**	0.074
0.029	0.214	**0.12**	0.445	**0.44**	0.279	**1.9**	0.071
0.03	0.216	**0.13**	0.458	**0.45**	0.273	**2**	0.068
0.032	0.221	**0.133**	0.461	**0.46**	0.267	**2.2**	0.061
0.035	0.228	**0.14**	0.468	**0.48**	0.257	**2.4**	0.055
0.036	0.231	**0.15**	0.476	**0.5**	0.248	**2.5**	0.052
0.04	0.241	**0.16**	0.476	**0.55**	0.223	**2.6**	0.05
0.042	0.246	**0.17**	0.474	**0.6**	0.203	**2.8**	0.045
0.044	0.251	**0.18**	0.472	**0.65**	0.185	**3**	0.042
0.045	0.254	**0.19**	0.469	**0.667**	0.18	**3.2**	0.038
0.046	0.256	**0.2**	0.466	**0.7**	0.171	**3.4**	0.035
0.048	0.262	**0.22**	0.444	**0.75**	0.158	**3.5**	0.034
0.05	0.267	**0.24**	0.423	**0.8**	0.148	**3.6**	0.033
0.055	0.284	**0.25**	0.413	**0.85**	0.14	**3.8**	0.03
0.06	0.3	**0.26**	0.403	**0.9**	0.134	**4**	0.028
0.065	0.317	**0.28**	0.386	**0.95**	0.129	**4.2**	0.027
0.067	0.323	**0.29**	0.377	**1**	0.124	**4.4**	0.026
0.07	0.333	**0.3**	0.369	**1.1**	0.111	**4.6**	0.025
0.075	0.348	**0.32**	0.354	**1.2**	0.104	**4.8**	0.024
0.08	0.362	**0.34**	0.34	**1.3**	0.097	**5**	0.023

* Envelope data for this site is a combination of the Minimum Deterministic Spectra and Probabilistic Spectra

9.3 Liquefaction Evaluation

Liquefaction can occur when saturated, loose to medium dense, granular soils (generally within 50 feet of the surface), or specifically defined cohesive soils, are subjected to ground shaking. Rock is present at shallow depths throughout the project area; therefore, we consider the potential for liquefaction of soils to be nonexistent at the OC.

9.4 Seismic Settlement

During a seismic event, ground shaking can cause densification of granular soil above the water table that can result in settlement of the ground surface. Rock is present at shallow depths throughout the project area; therefore, the potential for significant seismic settlement is low.

9.5 Seismic Slope Instability

Due to the presence of shallow rock and favorable rock structure, we consider the potential for seismic slope instability in the form of landslides and mudslides at this site to be very low. Similarly, we consider the potential for seismically induced rockslides or rockfall on engineered cut/fill slopes constructed at 1.5:1(horizontal: vertical) to be low. We present further slope stability evaluation below in the Foundation Recommendations.

10 FOUNDATION RECOMMENDATIONS

The most appropriate foundation type for the OC structure appears to be shallow spread footings established within moderately weathered rock at least 6 feet below existing grade at the abutments and 4 feet at the bent.

We considered Cast in Drilled Hole (CIDH) pile foundations or large diameter drilled-shafts and these are feasible but we expect difficult drilling due to both the hardness of the rock and the frequency of fractures. Driven piles are not an appropriate foundation alternative since they would experience very hard driving within rock at shallow depths (likely resulting in damage to the pile) and likely would not achieve adequate penetration for stability.

The General Plan and Foundation Plan for the project is in Appendix B. The following summarizes the proposed foundation design, as provided by MTCo:

- The foundation for Abutments 1 and 3 consist of spread footings. The footings are 9 to 11.5 feet wide, 106 feet long at Abutment 1, 109 feet long at Abutment 3, and 2.5 feet thick. The design requires a maximum contact pressure of approximately 4 kips per square foot (ksf).
- At Bent 2, MTCo proposes four columns, each with an individual spread footing. The footings are 13.5 feet square, 3.5 feet thick, with edges approximately 10.5 feet apart (24 feet on-center). The design requires a maximum contact pressure of approximately 21 ksf.

- Footings for Abutments 1 and 3 have split elevations. The planned bottom of Abutment 1 is 726.5 ft for the east half and 723.5 ft for the west half (approximately 22 to 24 feet below existing grade). Planned bottom of Abutment 3 is 727.5 ft for the east half and 731.5 ft for the west half (approximately 23 to 30 feet below existing grade).
- Footings for Bent 2 are at planned elevation 714.5, which is about 5.5 to 8.5 feet below existing grade.

10.1 Shallow Foundations

10.1.1 Spread Footing Data Table

Based on footing foundation design data provided by MTCo and our geotechnical analysis, we provide foundation design recommendations in Table 4. We include spread footing design calculations in Appendix D. A discussion of our analyses follows.

Table 4 – Foundation Design Recommendations for Spread Footings [1, 2]

Support Location	Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth (ft)	WSD (LRFD Service-I Limit State Load Combination)		LRFD		
							Service	Strength $\varphi_b = 0.45$	Extreme Event $\varphi_b = 1.0$
	B	L			Permissible Gross Contact Stress (ksf)	Allowable Gross Bearing Capacity (ksf)	Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Abut 1 - East	9.0	53.0	726.5[3]	6	30	30	N/A	N/A	N/A
Abut 1 - West	11.5	53.0	723.5[3]	6	30	30	N/A	N/A	N/A
Bent 2	13.5	13.5	714.5	4	N/A	N/A	30	22.5	50
Abut 3 - East	11.5	56.50	727.5[3]	6	30	30	N/A	N/A	N/A
Abut 3 - West	11.5	53.0	731.5[3]	6	30	30	N/A	N/A	N/A

Notes: 1) Recommendations are based on the foundation geometry and loads provided by the Design Engineer. The footing contact area is taken as equal to the effective footing area, if applicable.
2) See Memo to Designers (MTD) 4-1 for definitions and applications of the recommended design parameters.
3) Footing elevation conforms to MTCo Foundation Plan; higher levels may be acceptable for support if Phase 2 surface geometry allows

10.1.2 Slope Stability

The abutments will be founded behind a cut slope within metavolcanic rock. Maximum proposed slope gradients are 1.5(H):1(V) in front of both abutments. The finished maximum slope height ranges from 18 to 22 feet with up to 11 feet of height below abutment foundation elevation. Based on our slope stability assessment, we expect slopes below the abutments to be grossly stable at the proposed gradient and planned foundation configuration.

Rock slope stability is typically controlled by failure along discontinuities within the rock mass; however, weak rock masses can also fail through general shear. General shear failure could occur in cut slopes that expose weathered rock and have foundation loading. We evaluated the cut slopes below the abutments for stability on specific discontinuities and as a rock mass.

We recorded prominent discontinuities exposed at the existing cut slope and in nearby exploratory trenches. To evaluate the potential for slope failure along these discontinuities, we plotted them on a stereonet using the computer program ROCKPACK III. We show plots of these discontinuities in Appendix D. The plots show dip vectors (dip direction and magnitude) and planes (Great Circles) for the pervasive discontinuities we recorded.

We reviewed each plot for potential plane or wedge type failure. The plots show recorded discontinuities generally have a steep dip (greater than 60º), are mostly perpendicular to the proposed cut slope, and do not create planes or wedges out of slopes at a gradient of 1.5H:1V. At the north abutment cut slope (Abutment 3), there are possible minor wedges out of slope, plunging to the southwest, that occur on semi-continuous fractures with a shallow dip and some very steep fractures. The possible wedges plunge at low angles (16º or less) and are stable based on our review of wedge stability. Additionally, two planes dip obliquely out of slope at relatively low angles (16º to 25º) and, as potential plane failures, are considered stable based on the low dip angle, over 20º strike difference with the slope, and the tight/rough discontinuity surfaces. Based on recorded discontinuities, the proposed cut slopes will be grossly stable with respect to wedge and plane failures.

For rock mass stability, we completed a limit equilibrium analysis with the computer program SLIDE 6.0. We evaluated the 1.5H:1V slopes with abutment foundation loading and use material strengths and weights based on our laboratory tests (unconfined compressive strength), rock type, and estimates of rock mass strength based on the Generalized Hoek-Brown failure criteria. Rock mass strength is based on rock type, quality, and weathering (Practical Rock Engineering, Hoek, 2006). We conservatively estimate an intensely to moderately weathered rock mass for the full slope height and use the following rock mass properties:

- Intact Unconfined Compressive Strength = 5,000 psi (720,000 psf)
- Rock Unit Weight = 170 pcf
- Geological Strength Index = 45
- Hoek-Brown constant (m_i) = 12

Our analyses indicate a Factor of Safety (Spencer Method) greater than 2.5 under static loading (seismic loading is not analyzed since Factor of Safety is greater than 1.7), which indicates that the slope will be stable at the proposed gradient (1.5H:1V) with foundation loading included. Maximum foundation loads of 20 ksf are included as an applied load at foundation level (actual loads will be less than 5 ksf). We show the sections analyzed in Appendix D.

Excavation for mainline widening in front of Abutment 3 will occur in two phases. Limited slope excavation will occur during Phase 1, which will leave significant excavation for Phase 2. Rock excavation at these slopes may require drilling/splitting and or chiseling; the Phase 1 excavation must provide adequate access/clearance for equipment during Phase 2 excavation. Complete significant, if not all, excavation of the final slope during Phase 1 to avoid future excavation difficulties and possible impacts to the abutment foundation materials.

10.1.3 Lateral Resistance

Calculate lateral load resistance of spread footings as follows:

- A soil friction factor (tan δ) of 0.45 for cast in-place concrete foundations bearing on intact rock materials or compacted structure backfill. Use a resistance factor (φ_τ) of 0.8 for LRFD.
- An allowable passive pressure of 270 pcf equivalent fluid pressure against the face of the footing (based on formed footings with compacted structure backfill or footings poured neat against intact rock); neglect the upper 3 feet of soil depth (from final ground surface) in determination of passive earth pressure due to potential soil disturbance/removal. Use a resistance factor (φ_{ep}) of 0.5 for LRFD.
- Passive and friction resistance may be combined.

If necessary for increased sliding resistance, use steel rock dowels with minimum diameter of 1¼-inch (#9 bars) grouted in drilled holes at least 5 feet into rock. Maintain a minimum spacing of at least 3-feet (center-to-center) between dowels.

10.1.4 Settlement

Based on the proposed design loads and the underlying rock conditions, total settlement at abutment and bent foundations will not exceed ½-inch. We do not expect differential settlement between adjacent footings to exceed ½-inch. Total settlement of spread footing foundations at the abutments and bents is based on empirical values for footings on competent rock (Caltrans BDS 4.4.8.2).

10.2 Retaining Walls

A retaining wall/wingwall, Caltrans Standard Type 1, is located on the east side of Abutment 3. The wall is 26 feet long and varies in design height from 14 to 18 feet. Foundations step up behind the abutment from elevation 732.5 to 737.0 feet.

For a Type 1 wall with level backfill (Case I) condition, Caltrans "Standard Plans" (2010) show maximum toe pressures of 2.6 ksf to 3.1 ksf (Strength Limit) for retaining wall heights between 14 feet and 18 feet in design height.

We expect the planned retaining wall to engage hard, slightly to moderately weathered rock. Excavation of the adjacent abutment foundation (at elevation 727.5 ft) may create the need for additional forming and concrete fill below the planned foundation depth.

Adequate bearing capacity (in excess of 20 ksf at the factored strength limit) is available for the proposed Caltrans Type 1 retaining wall foundations established within rock at the planned foundation elevations. Predominant rock structure (discontinuities such as foliation and fracture) at Abutment 3 has a steep dip (greater than 60º to the northeast) that is favorable for future ramp cuts (Phase 2) adjacent to this wall. We do not expect the future ramp cuts to affect wall stability; however, review this condition following Phase 1 construction and prior to Phase 2 construction based on the final slope configuration.

Maximum and differential settlement across and along the walls will be less than 1-inch. Due to the presence of the underlying rock, we expect settlement to be minimal and occur substantially during construction.

10.3 Approach/Abutment Backfill Earthwork

10.3.1 Fill Material

The Abutment locations will be cut to grade; therefore, placement of significant approach fill is not expected. Locally excavated materials are expected for use as approach fill. Any proposed borrow must be tested and approved for use by the project engineer prior to transporting to the site.

10.3.2 Expansive Material

Expansive materials shall not be placed as part of the embankment within the limits of the bridge abutment for the full width of the embankment. Low expansion material is defined as having an Expansion Index (EI) less than 50 (per ASTM D4829), and a Sand Equivalent (SE) greater than 20 (per California Test 217).

10.3.3 Geometry and Stability

Where approach fill is placed, side slopes will have a gradient of 2H:1V or flatter. The slope facing US 50 will have a gradient of 1.5H:1V with slope paving.

The proposed geometries are common slope gradients considered stable for typical approach fill construction. We assume backfill will consist of materials conforming to Structure Backfill requirements. The mostly flat nature of the existing ground surface and high strength of the underlying rock will provide a stable base on which to construct the fills.

10.3.4 Site Preparation

In the area of approach fills, clear and grub existing slopes in accordance with the Caltrans “Standard Specifications”, Section 16. Construct structure backfill at the abutments in accordance with the “Standard Specifications”, Section 19-3.06. Construct the embankment approach fills in accordance with the “Standard Specifications”, Section 19-6.01.

10.3.5 Settlement

Due to the presence of shallow rock, we do not anticipate significant settlement at approaches. We expect post-construction settlement between the abutment backwall and adjacent approach fills/backfill to be less than ½-inch, provided structure backfill is compacted in accordance with the Caltrans “Standard Specifications.” A waiting period is not necessary.

10.3.6 Lateral Earth Pressures

We assume that the approach fill material meets the requirements of Caltrans standard for Structure Backfill. Use the following equivalent fluid weights (EFW) to design the abutments walls and wing walls at Abutments 1 and 3:

Condition	EFW Static	EFW Seismic
Active	36 lb/ft^3	4 lb/ft^3
At-Rest	55 lb/ft^3	7 lb/ft^3
Passive	270 lb/ft^3	250 lb/ft^3

For static design, apply the resultant of the static active earth pressure (36 lb/ft^3) at a distance of 0.33H above the base of the wall where H equals the wall height in feet.

For seismic design, calculate the resultant of incremental lateral soil pressure due to seismic loading based on an equivalent fluid pressure of 4 lb/ft^3 for active condition and 7 lb/ft^3 for at-rest condition. Apply the magnitude of the resultant seismic active and at-rest pressures at 0.5H from the base of the wall. Add the resultant of the seismic earth pressure to the resultant of the static earth pressure.

The values shown above are consistent with Caltrans standards/practice and assume level backfill conditions using Caltrans “Structure Backfill” with a soil unit weight of 120 pcf, a minimum angle of internal friction of 33º, and that wall drainage is placed in accordance with Caltrans “Standard Plans and Specifications.”

To limit wall deflection to acceptable levels, BCI applied a factor of safety of 2.0 to the ultimate passive pressure to generate the allowable passive pressures provided above.

BCI estimated the EFWs for seismic loading using the Mononobe-Okabe equation for active and passive lateral coefficients K_a and K_p. We estimated the at-rest coefficient, K_o, for the seismic condition using an increase ratio similar to the active condition. In the Mononobe-Okabe equation, BCI used a horizontal seismic acceleration coefficient (k_h) of 0.10 calculated using the equation in Chapter 11, Section 11.6.5 of the AASHTO LRFD Bridge Design Specifications-4th Edition. This k_h value assumes that the walls displace at least 1-inch during the design seismic event. BCI calculated the above static EFWs using methods presented in the 1982 Naval Facilities (NAVFAC) Design Manual 7.2.

For seismic loading into abutments, use a maximum passive pressure of 5.0 ksf for longitudinal abutment response, with the proportionality factor presented in Section 7.8.1 of Caltrans Seismic Design Criteria v.1.6 (November 2010).

For surcharge loads, apply an additional uniform lateral load behind the wall equivalent to 0.3-times the surcharge pressure. Use a soil friction factor (tan δ) of 0.45 for cast in-place concrete foundations bearing on weathered rock or compacted fill materials.

11 CONSTRUCTION CONSIDERATIONS

11.1 Cuts and Excavations

Typical grading equipment such as scrapers, dozers, backhoes and excavators are sufficient to excavate surficial soil and decomposed to intensely weathered rock at the proposed overcrossing. However, due to the presence of moderately hard to hard rock (particularly at the bent foundation locations), foundation excavation may require a large excavator equipped with rock teeth and a single-shank rock ripper attachment. Use of air tools (chiseling and rock splitting) will likely be required at the bent foundation locations and isolated abutment foundation locations.

Temporary slopes may be required for foundation construction. The Contractor shall slope and/or shore temporary excavations in accordance with current Cal-OSHA requirements. Where the use of excavation sloping and/or shoring is required, a competent person must classify each soil deposit as Type A, Type B, or Type C in accordance with OSHA procedures, and shall confirm the soil types during construction. Based on our investigation, we preliminarily classify native soils as Type B. Design excavation sloping and/or shoring located in any fill material in accordance with Type C soils.

At bridge support and wall locations, rock blasting may disrupt/degrade integrity of the surrounding rock. Therefore, rock blasting should not be permitted to construct new bridge foundations. If it is required, remove all overblast and/or shattered rock prior to placement of reinforcement and concrete.

Large blocks may pull-out from walls of foundation excavations. Fill any cavities, at and below foundation level, formed by the blocks with structural concrete.

Excavation for mainline widening in front of Abutment 3 is planned in two phases. Limited slope excavation will occur during Phase 1, which will leave significant excavation for Phase 2. Rock excavation will likely require drilling/splitting and or chiseling; Phase 1 excavation must provide adequate access/clearance for equipment during Phase 2 excavation. Complete significant, if not all, excavation of the final slope during Phase 1 to avoid future excavation difficulties and possible impacts to the abutment foundation materials. Rock blasting should not be permitted for slope excavation below the abutment/wall foundations.

11.2 Embankments

Import borrow sources are not yet identified and must be evaluated and approved for use as embankment fill prior to transporting or use. We expect slopes constructed of on-site materials or imported borrow to meet the specifications for embankment fill and, sloped at a gradient of 2H:1V or flatter, to be grossly stable. Material used for backfill at abutments must meet the requirements for Structure Backfill.

11.3 Spread Footings

Pour footing concrete "neat" (without forming), against trimmed, intact bearing material within clean and dry excavations. If forming is necessary, backfill excavations outside footing limits with lean concrete or suitable granular backfill (i.e. "Structure Backfill" per Caltrans "Standard Specifications") compacted to at least 95% relative compaction (per CTM 216).

If it is necessary to deepen footing excavations in order to engage suitable bearing materials, it is acceptable to backfill with structural concrete to plan footing grade, up to a depth of 3 feet below the footing, with BCI approval. Conversely, to avoid excessive excavation, stepping of footings is acceptable to achieve required penetration of bearing materials.

BCI's representative must review foundation excavations for suitable bearing material and evaluate. Review open joint/fractures exposed in foundation excavations with respect to bearing/stability considerations and clean/surface-grout if necessary.

11.4 Dewatering

We do not anticipate the presence of significant groundwater within footing excavations during dry season construction (June through October). If/where seepage is encountered, we expect it can be controlled with sump pumps. Winter or spring construction may encounter perched groundwater, possibly under head, and require additional controls.

11.5 Naturally Occurring Asbestos

During our site reconnaissance and subsurface exploration we did not observe outcrops containing serpentinite or other ultramafic rock, a host rock for naturally occurring asbestos minerals (NOA), or significant bands of fibrous (asbestiform) minerals within the visible bedrock. As discussed above, NOA mapping does not show the project within an ultramafic rock area, although the project is near mapped faults and other areas known to contain naturally occurring asbestos. We cannot rule out the potential for NOA to occur at the project site and it will need to be considered as a potential risk during construction.

BCI recommends preparation of an Asbestos Hazard Mitigation Plan in compliance with provisions of El Dorado County Air Quality Management District (EDAQMD) Rule 223-2.and California Air Resources Board requirements, as applicable.

Visually monitor rock types exposed during construction for the potential presence of naturally occurring asbestos (NOA) minerals. If excavations expose NOA, comply with the applicable provisions of EDAQMD Rule 223-2 and the State of California Asbestos Airborne Toxic Control Measure (ACTM), CCR Title 17, Section 93105. In addition, prepare a worker health and safety program for excavations in areas with NOA in accordance with all regulatory requirements, including CAL OSHA.

11.6 Storm Water Quality

We expect that construction term erosion control will be available by means of typical good construction practices (e.g., use of erosion barriers, synthetic slope covers, hydro-seeding, etc.). This project will involve earthwork and we expect that the contractor will be required to develop a Storm Water Pollution Prevention Plan.

12 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services. For this project, retain BCI to:

- Review and provide written comments on the (civil, structural) plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, we should review footing excavations and cut slopes, and observe and test fill construction.
- Update this report if design changes occur, 2 years lapse between this report and construction, or site conditions change.

If BCI is not retained to perform the above applicable services, we are not responsible for any other parties' interpretation of our report, and subsequent addendums, letters, and discussions.

13 LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. We do not warranty our services.

BCI based this report on the current site and project conditions. We assumed the soil, rock, and groundwater conditions we observed in our borings are representative of the subsurface conditions on the site. Actual conditions between borings could be different.

Use this foundation report only for the design and construction of the US 50 / Silva Valley Parkway Overcrossing.

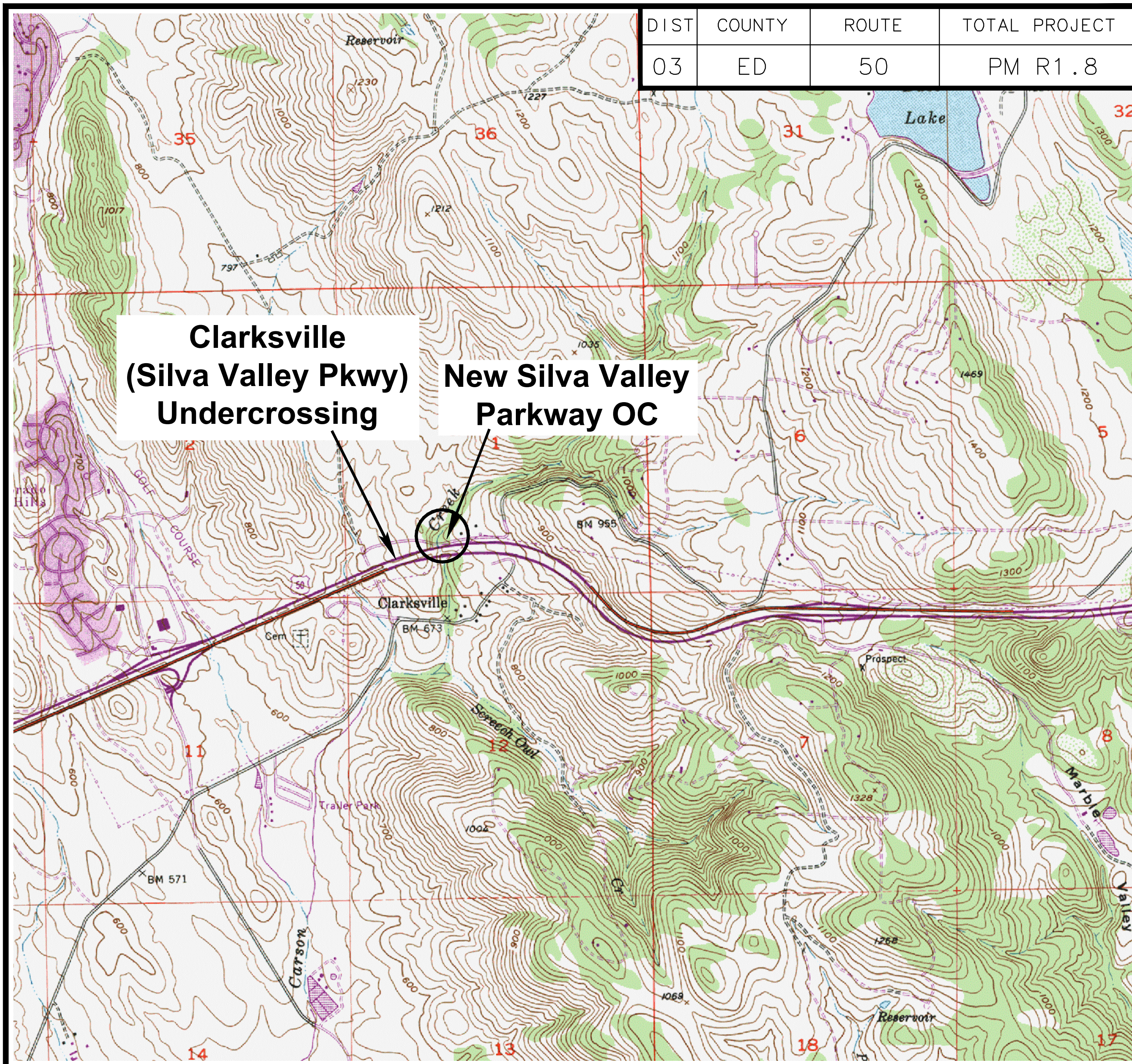
Modern design and construction is complex, with many regulatory sources, restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

The interface between soil and rock materials on the logs is approximate. The transition between materials may be abrupt or gradual. We base our recommendations on the final logs, which represent our interpretation of the field logs and general knowledge of the site and geological conditions.

APPENDIX A

Figure 1 – Vicinity Map
Figure 2 – Regional Geologic Map
Figure 3 – Seismic Hazard Map
Figure 4 – ARS Curve

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.8



N

Source: MAPTECH Terrain Navigator Pro, v. 7.01, USGS topographic map, 7.5 minute quadrangle, 1:24000, Clarksville 1953 (revised 1980).

SCALE: 1"=0.5 Miles

blackburn consulting

11521 Blocker Drive, Suite 110
Auburn,CA 95603
Phone: (530) 887-1494
Fax: (530) 887-1495
www.blackburnconsulting.com

VICINITY MAP

Silva Valley Parkway Overcrossing
EA 03-1E2901
El Dorado County, California

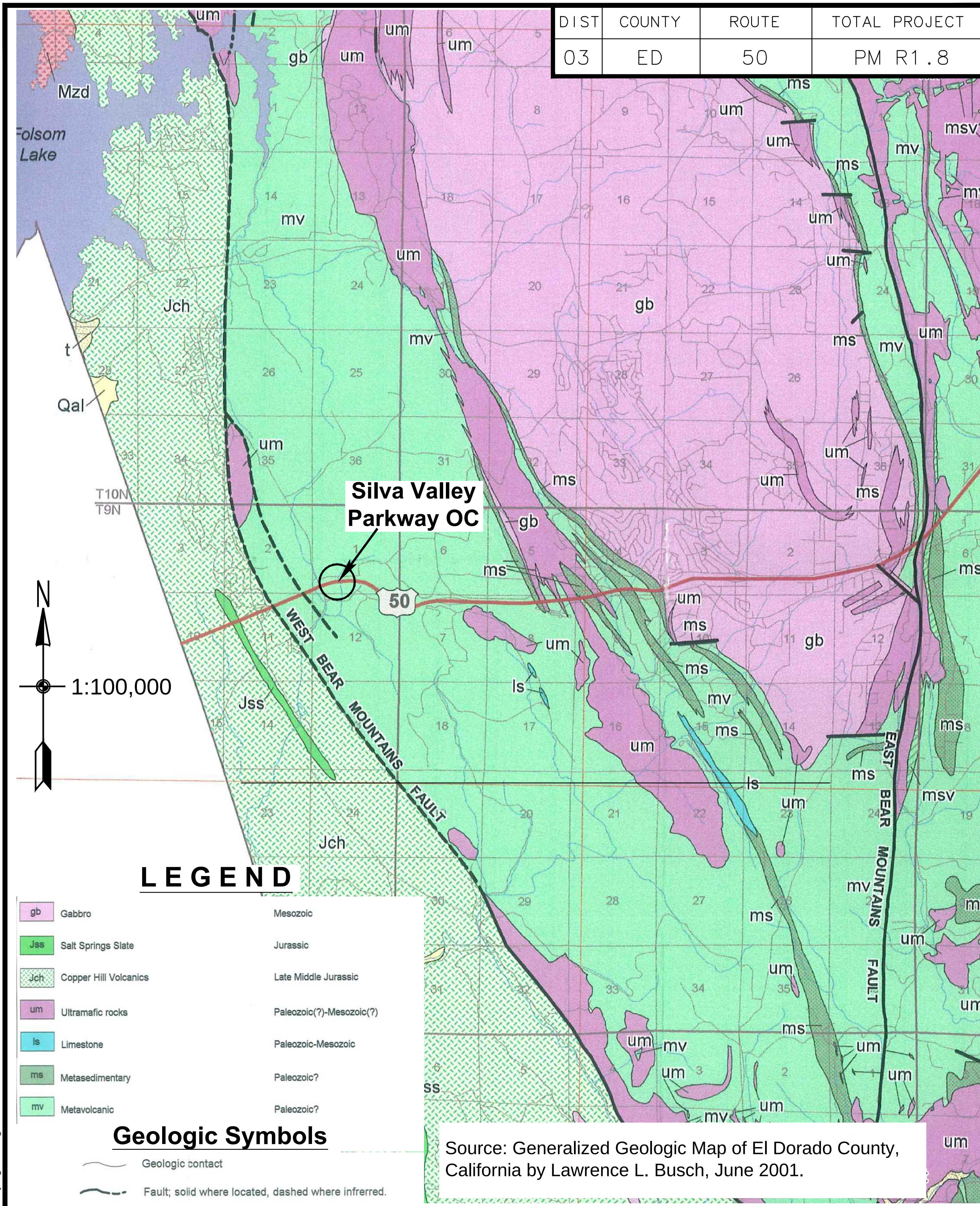
File No. 556.2

April 2012

Figure 1

4/9/2012 556.2 Silva Valley Parkway Figure 1.dwg

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.8



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Fax: (530) 887-1495
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GEOLOGIC MAP

Silva Valley Parkway Overcrossing
EA 03-1E2901
El Dorado County, California

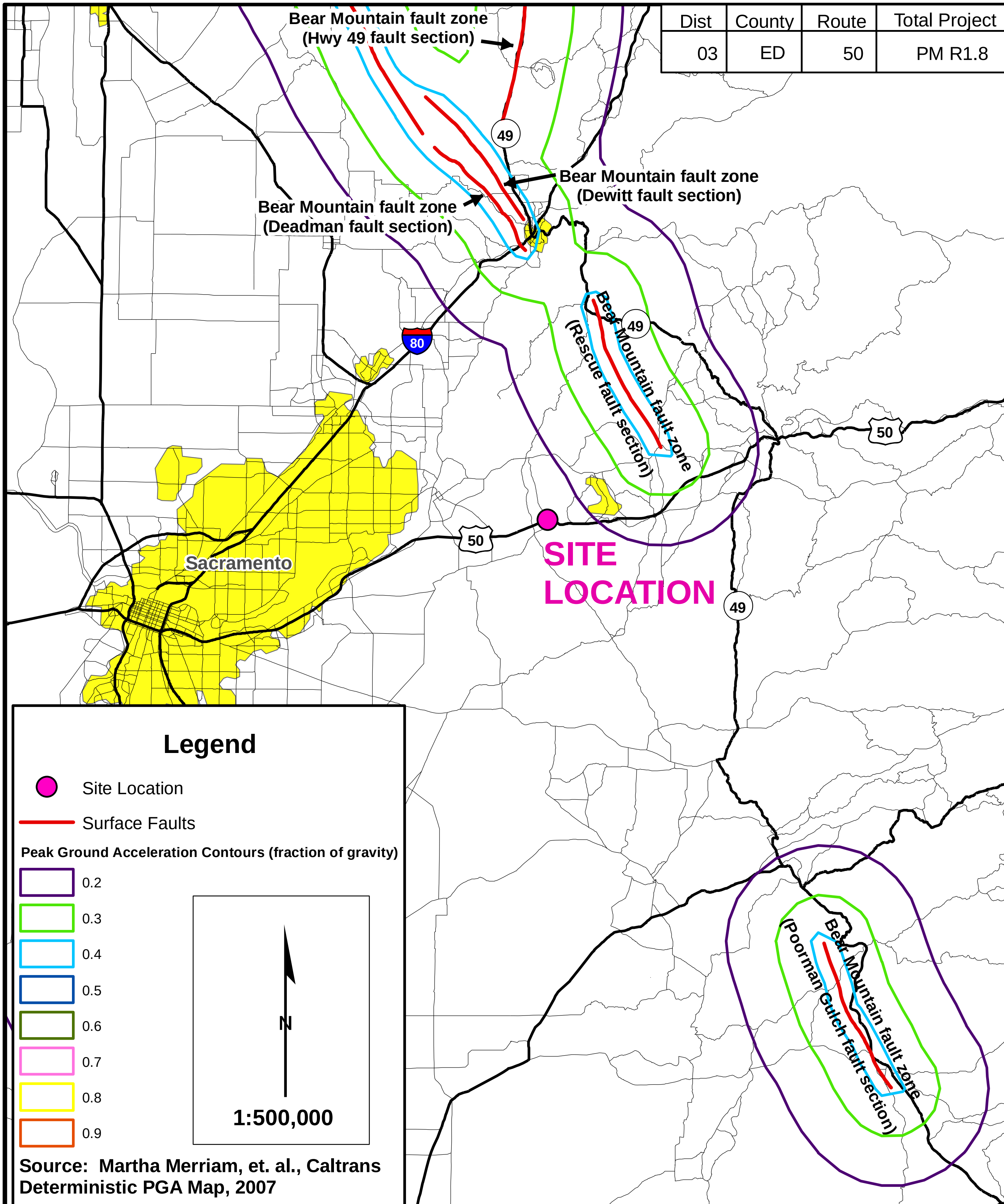
File No. 556.2

April 2012

Figure 2

4/9/2012 556.2 Silva Valley Parkway Figure 2.dwg

Dist	County	Route	Total Project
03	ED	50	PM R1.8



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SEISMIC HAZARD MAP

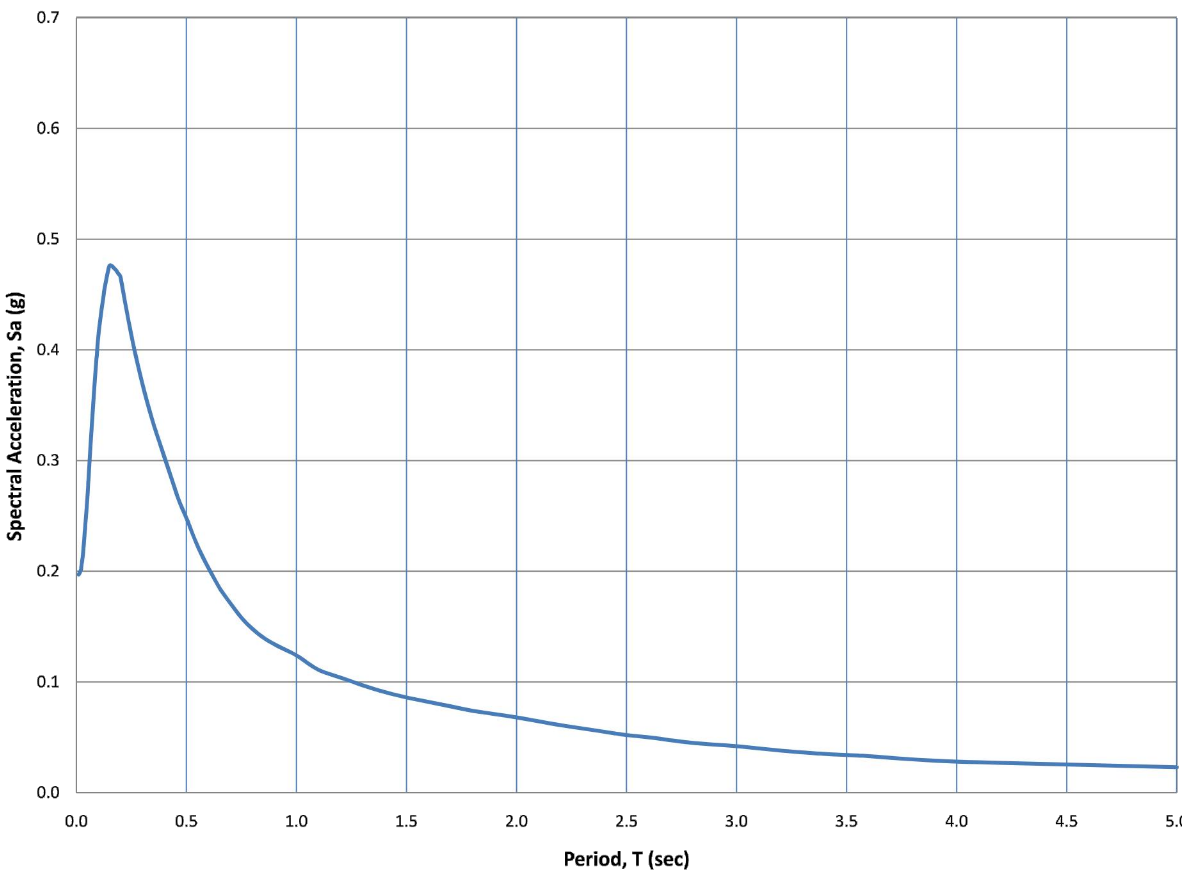
Silva Valley Parkway Overcrossing
EA-03-01E2901
El Dorado County, California

File No. 556.2

April 2012

Figure 3

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.8



Caltrans ARS Online (V1.04)

blackburn consulting

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ARS CURVE

Silva Valley Parkway Overcrossing
EA 03-1E2901
El Dorado County, California

File No. 556.2

April 2012

Figure 4

4/9/2012 556.2 Silva Valley Parkway Figure 4.dwg

APPENDIX B

Log of Test Borings (4 sheets)
General Plan (MTCo)
Foundation Plan (MTCo)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.06/2.90		

CERTIFIED ENGINEERING GEOLOGIST DATE

PROFESSIONAL GEOLOGIST PATRICK F. FISCHER No. 1739 Exp. 1/31/13 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

PLANS APPROVAL DATE

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BLACKBURN CONSULTING
11521 BLOCKER DRIVE, SUITE 110
AUBURN, CA 95603 FILE No. 556.2

MARK THOMAS & CO., INC.
7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

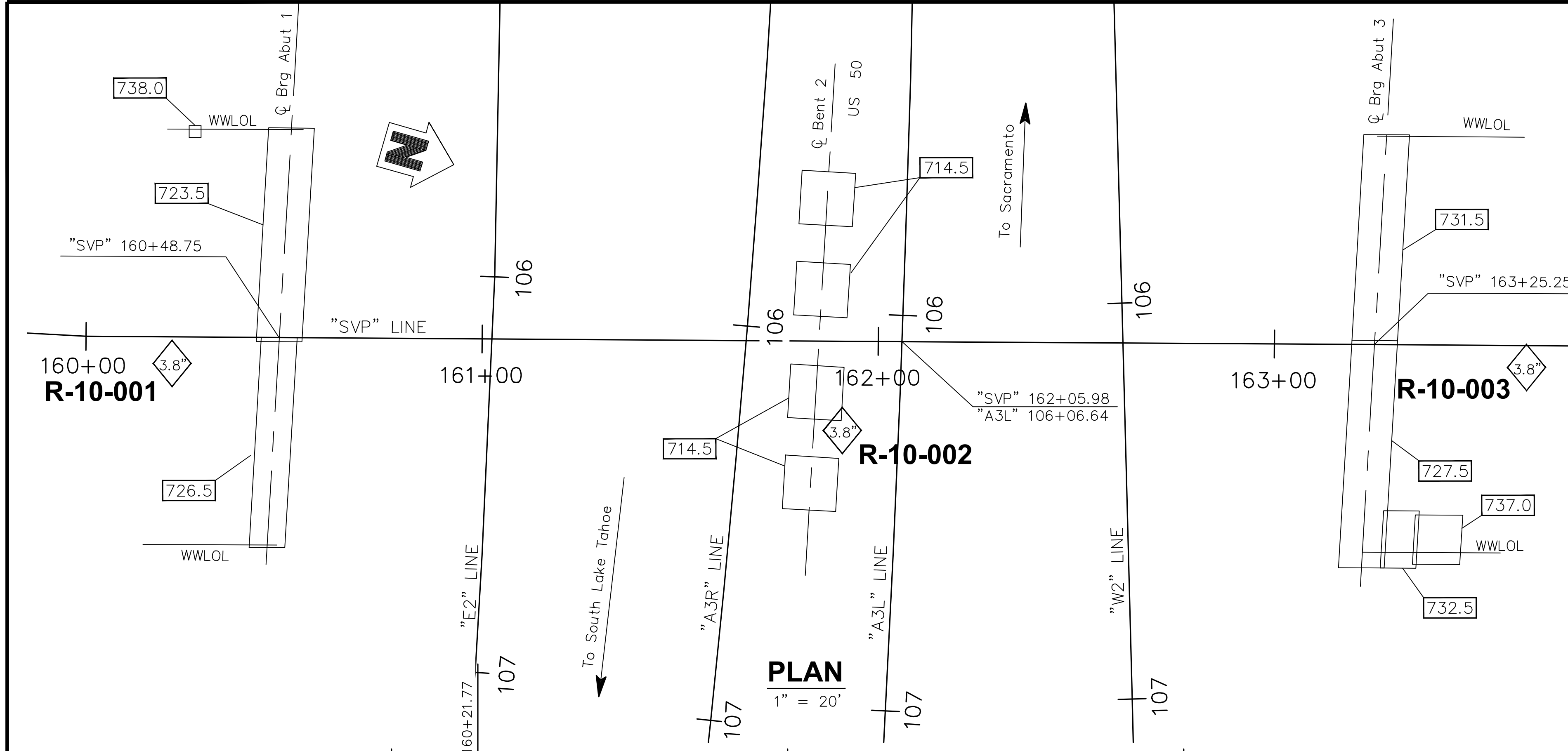
NOTES:

1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2007). See Log of Test Borings No. 2, and 3, "Soil Legend" and 4, "Rock Legend".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with brass liners.
3. "2.4 inch sampler": ID=2.4 inch, OD=2.9 inch. Driven in same manner as SPT ("1.4 inch") sampler.
4. Where less than the 0.5 inches of penetration is achieved, the blow count shown is for that fraction of the interval actually penetrated.
5. Where indicated by an asterisk (*) the number of blows shown is for only that fraction of the initial 0.5 ft. "seating drive" interval penetrated.
6. If laboratory tests are not shown as being performed, the soil descriptions presented in the LOTB are based solely on the visual practices described in the before mentioned Manual.
7. The length of each sampled interval is shown graphically on the boring log.
8. Consistency of soils shown in ( ) where estimated.
9. Groundwater surface (GWS) reflect the fluid level in the borings on the specified date. Groundwater surface is subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
10. Electronic media for plan view provided by Mark Thomas & Co., Inc., dated 05/2010.
11. Boring elevations are approximate and based on plans provided by Mark Thomas & Co., Inc.
12. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".

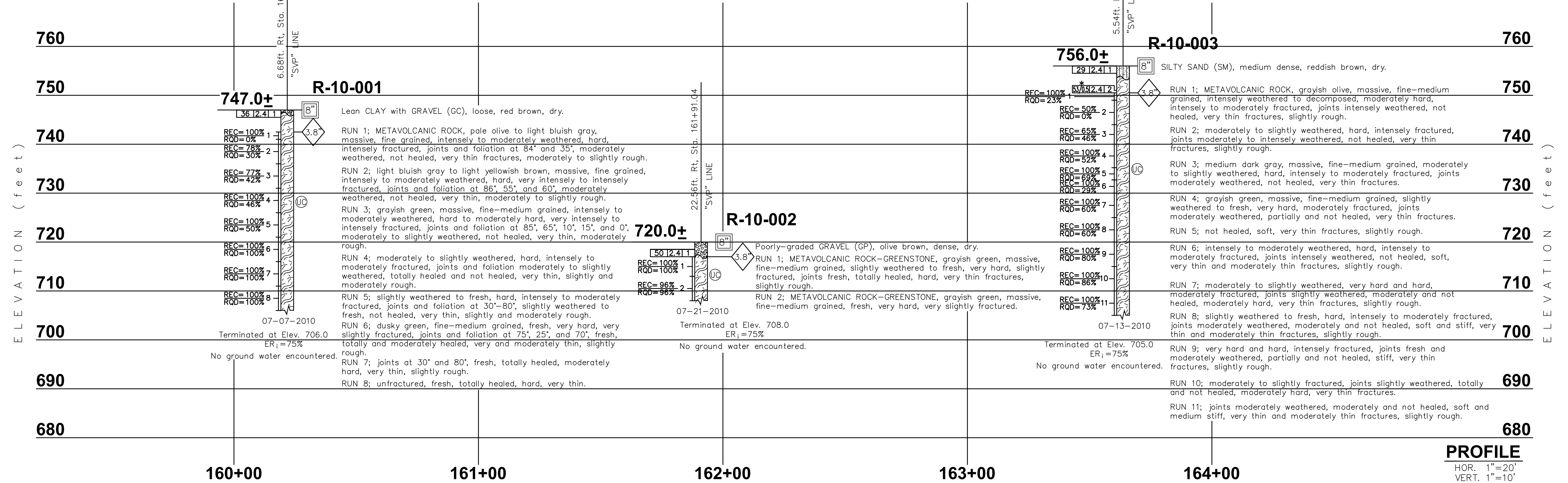
BENCHMARKS

Benchmark No. 1:
Orthometric Heights (elevations) shown are NGVD 29. Based on HPGN D CA 03 DL having an elevation of 693.55 feet.

Benchmark No. 2:
USGS BM T 127 (PID JS0692) having an elevation of 673.08 feet.



PLAN
1" = 20'



PROFILE
HOR. 1"=20'
VERT. 1"=10'

DESIGN OVERSIGHT	DRAWN BY	M. ROBERTSON	R. PICKARD — FIELD INVESTIGATION BY:
SIGN OFF DATE	CHECKED BY	R. PICKARD	DATE: July 2010

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0127
POST MILE R1.8

SILVA VALLEY PARKWAY OVERCROSSING
LOG OF TEST BORINGS 1 OF 4

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL, (JUNE, 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.06/2.90		

CERTIFIED ENGINEERING GEOLOGIST DATE

PLANS APPROVAL DATE

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7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	<0.25	<0.25	<0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

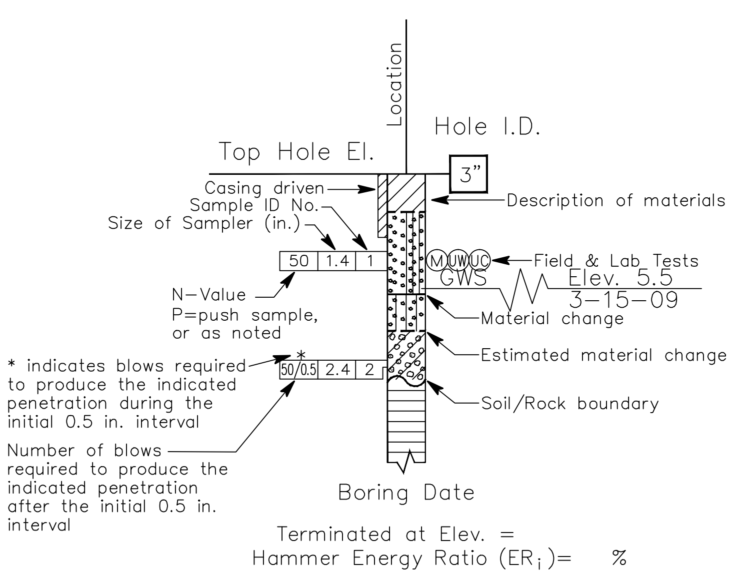
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other

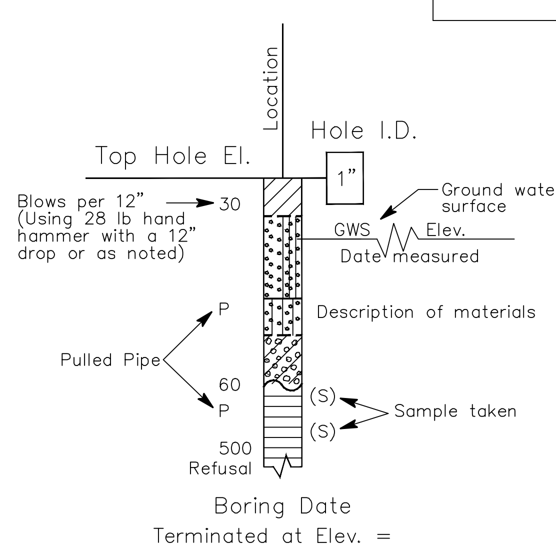
NOTE: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

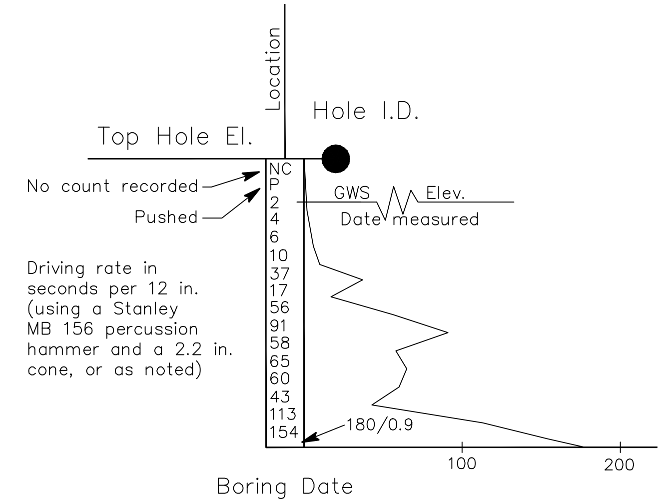
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



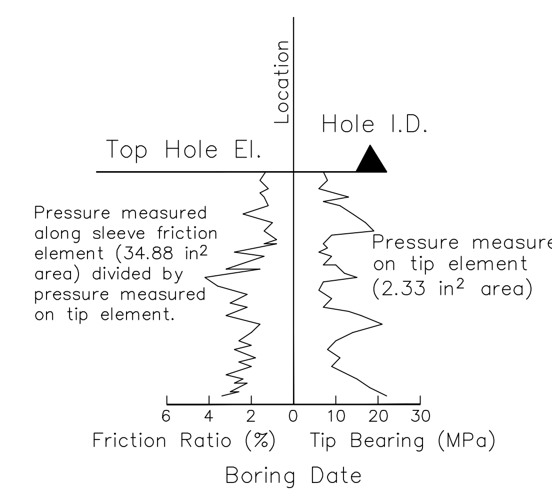
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

SOIL LEGEND

SILVA VALLEY PARKWAY OVERCROSSING

LOG OF TEST BORINGS 2 OF 4

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY M. ROBERTSON

CHECKED BY R. PICKARD

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0127
POST MILE R1.8

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REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL, (JUNE, 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND
GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL
SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean Clay ORGANIC lean Clay with SAND ORGANIC lean Clay with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OH/OL	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- C Consolidation (ASTM D 2435)
- CL Collapse Potential (ASTM D 5333)
- CP Compaction Curve (CTM 216)
- CR Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- CU Consolidated Undrained Triaxial (ASTM D 4767)
- DS Direct Shear (ASTM D 3080)
- EI Expansion Index (ASTM D 4829)
- M Moisture Content (ASTM D 2216)
- OC Organic Content-% (ASTM D 2974)
- P Permeability (CTM 220)
- PA Particle Size Analysis (ASTM D 422)
- PI Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- PL Point Load Index (ASTM D 5731)
- PM Pressure Meter
- PP Pocket Penetrometer
- R R-Value (CTM 301)
- SE Sand Equivalent (CTM 217)
- SG Specific Gravity (AASHTO T 100)
- SL Shrinkage Limit (ASTM D 427)
- SW Swell Potential (ASTM D 4546)
- TV Pocket Torvane
- UC Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- UU Unconsolidated Undrained Triaxial (ASTM D 2850)
- UW Unit Weight (ASTM D 2937)
- VS Vane Shear (AASHTO T 223)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60}-Value (Blows / 12 inches)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

SOIL LEGEND

SILVA VALLEY PARKWAY OVERCROSSING

LOG OF TEST BORINGS 3 OF 4

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY M. ROBERTSON

CHECKED BY R. PICKARD

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0127
POST MILE R1.8

GS LOTB SOIL LEGEND SHEET 2 (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS
0 1 2 3

UNIT: 03-1E2901
PROJECT NUMBER & PHASE: 0300000258
CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 04/05/12

FILE => $REQUEST

4/5/2012 556.2 Silva Valley Parkway LOTB.dwg

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.06/2.90		

CERTIFIED ENGINEERING GEOLOGIST DATE

PLANS APPROVAL DATE

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

PROFESSIONAL GEOLOGIST PATRICK F. FISCHER No. 1739 Exp. 1/31/13 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

BLACKBURN CONSULTING
11521 BLOCKER DRIVE, SUITE 110
AUBURN, CA 95603 FILE No. 556.2

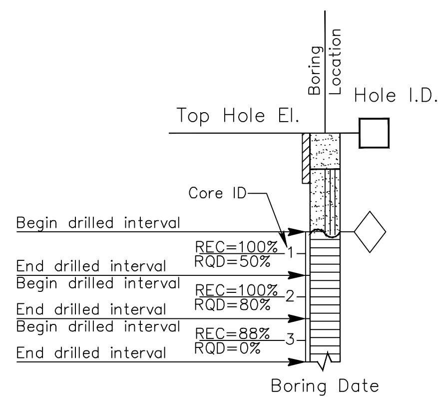
MARK THOMAS & CO., INC.
7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL, (JUNE, 2007)

PERCENT CORE RECOVERY (REC) & ROCK QUALITY DESIGNATION (RQD)

$$REC = \frac{\sum \text{Length of the recovered core pieces (inches)}}{\text{Total length of core run (inches)}} \times 100\%$$

$$RQD = \frac{\sum \text{Length of the intact core pieces} \geq 4"}{\text{Total length of core run (inches)}} \times 100\%$$



RELATIVE STRENGTH OF INTACT ROCK

Term	Uniaxial Compressive Strength (PSI)
Extremely Strong	> 30,000
Very Strong	14,500 – 30,000
Strong	7,000 – 14,500
Medium Strong	3,500 – 7,000
Weak	700 – 3,500
Very Weak	150 – 700
Extremely Weak	< 150

BEDDING SPACING

Description	Thickness / Spacing
Massive	Greater than 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3–5/8" to 1 ft
Thinly bedded	1–1/4" to 3–5/8"
Very thinly bedded	3/8" to 1–1/4"
Laminated	Less than 3/8"

LEGEND OF ROCK MATERIALS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

ROCK HARDNESS

Description	Criteria
Extremely Hard	Specimen cannot be scratched with a pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows.
Very Hard	Specimen cannot be scratched with a pocket knife or sharp pick. Breaks with repeated heavy hammer blows.
Hard	Specimen can be scratched with a pocket knife or sharp pick with difficulty (heavy pressure). Heavy hammer blows required to break specimen.
Moderately Hard	Specimen can be scratched with a pocket knife or sharp pick with light or moderate pressure. Core breaks with moderate hammer pressure.
Moderately Soft	Specimen can be grooved 1/16" deep with a pocket knife or sharp pick with moderate or heavy pressure. Breaks with light hammer blow or heavy manual pressure.
Soft	Specimen can be grooved or gouged easily by a pocket knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	Specimen can be readily indented, grooved or gouged with fingernail, or carved with a pocket knife. Breaks with light manual pressure.

FRACTURE DENSITY

Description	Observed Fracture Density
Unfractured	No fractures.
Very slightly fractured	Lengths greater than 3 feet.
Slightly fractured	Lengths from 1 to 3 feet with few lengths less than 1 foot or greater than 3 feet.
Moderately fractured	Lengths mostly in 4" to 1 foot range with most lengths about 8"
Intensely fractured	Lenghts average from 1" to 4" with scattered fragmented intervals with lengths less than 4 in.
Very intensely fractured	Mostly chips and fragments with a few scattered short core lengths.

Combination descriptors (such as "Very intensely to intensely fractured") are used where equal distribution of both fracture density characteristics is present over a significant interval or exposure, or where characteristics are "in between" the descriptor definitions. Only two adjacent descriptors may be combined.

WEATHERING DESCRIPTORS FOR INTACT ROCK

Description	Diagnostic features					General Characteristics
	Chemical weathering–Discoloration and/or oxidation		Mechanical Weathering–Grain boundary conditions (disaggregation) primarily for granitics and some coarse–grained sediments	Texture and solutioning		
	Body of rock	Fracture Surfaces		Texture	Solutioning	
Fresh	No discoloration, not oxidized.	No discoloration or oxidation.	No separation, intact (tight).	No change.	No solutioning.	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull.	Minor to complete discoloration or oxidation of most surfaces.	No visible separation, intact (tight).	Preserved.	Minor leaching of some soluble minerals may be noted.	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe–Mg minerals are "rusty", feldspar crystals are "cloudy".	All fracture surfaces are discolored or oxidized.	Partial separation of boundaries visible.	Generally preserved.	Soluble minerals may be mostly leached.	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe–Mg minerals are altered to clay to some extent; or chemical alteration produces in–situ disaggregation, see grain boundary conditions.	All fracture surfaces are discolored or oxidized, surfaces friable.	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated.	Texture altered by chemical disintegration (hydration, argillation).	Leaching of soluble minerals may be complete.	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hair–line fractures, or veinlets. Rock is significantly weakened.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe–Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated).	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete.		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic features. However, combination descriptors should not be used where significant, identifiable zones can be delineated. Only two adjacent descriptors may be combined. "Very intensely weathered" is the combination descriptor for "intensely weathered to decomposed".

ROCK LEGEND

SILVA VALLEY PARKWAY OVERCROSSING

LOG OF TEST BORINGS 4 OF 4

DESIGN OVERSIGHT	DRAWN BY	M. ROBERTSON
SIGN OFF DATE	CHECKED BY	R. PICKARD

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25–0127
POST MILE R1.8

GS LOTB ROCK LEGEND SHEET 1 (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 03–1E2901
PROJECT NUMBER & PHASE: 0300000258 CONTRACT NO.: X

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 04/05/12 SHEET 27 OF 27

FILE => $REQUEST

4/5/2012 556.2 Silva Valley Parkway LOTB.dwg

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.06/2.90		

REGISTERED STRUCTURAL ENGINEER DATE

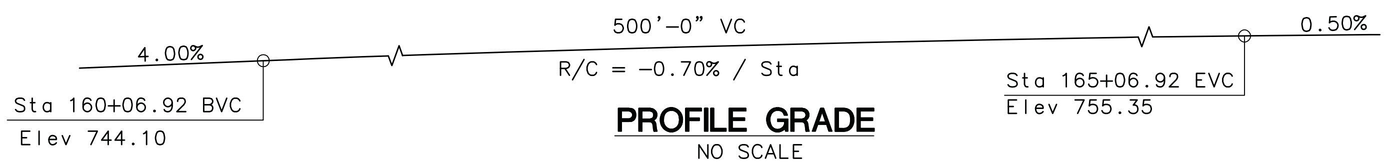
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER PO-KANG CHEN No. S3112 Exp. 9/30/13 STRUCTURAL STATE OF CALIFORNIA

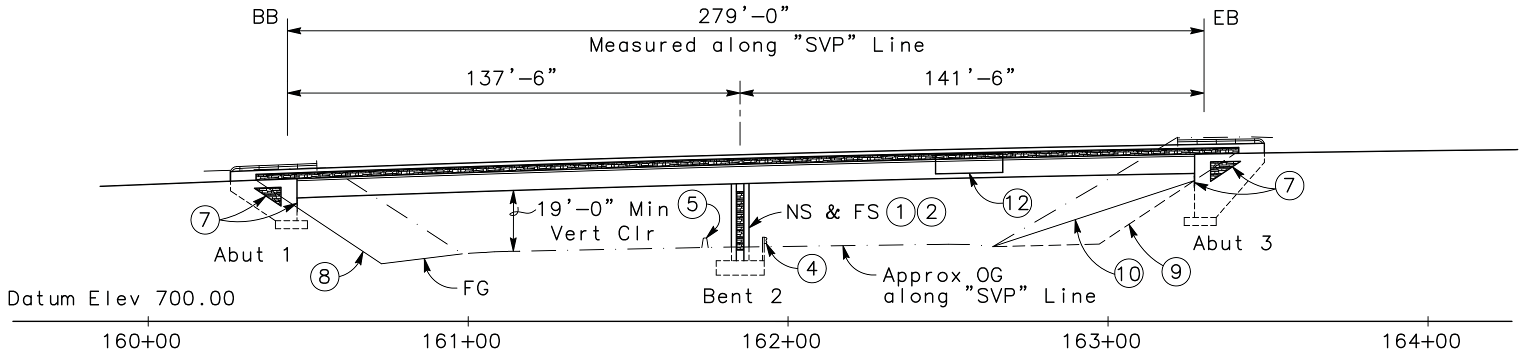
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
4950 HILLSDALE CIR STE 400
EL DORADO HILLS, CA 95762

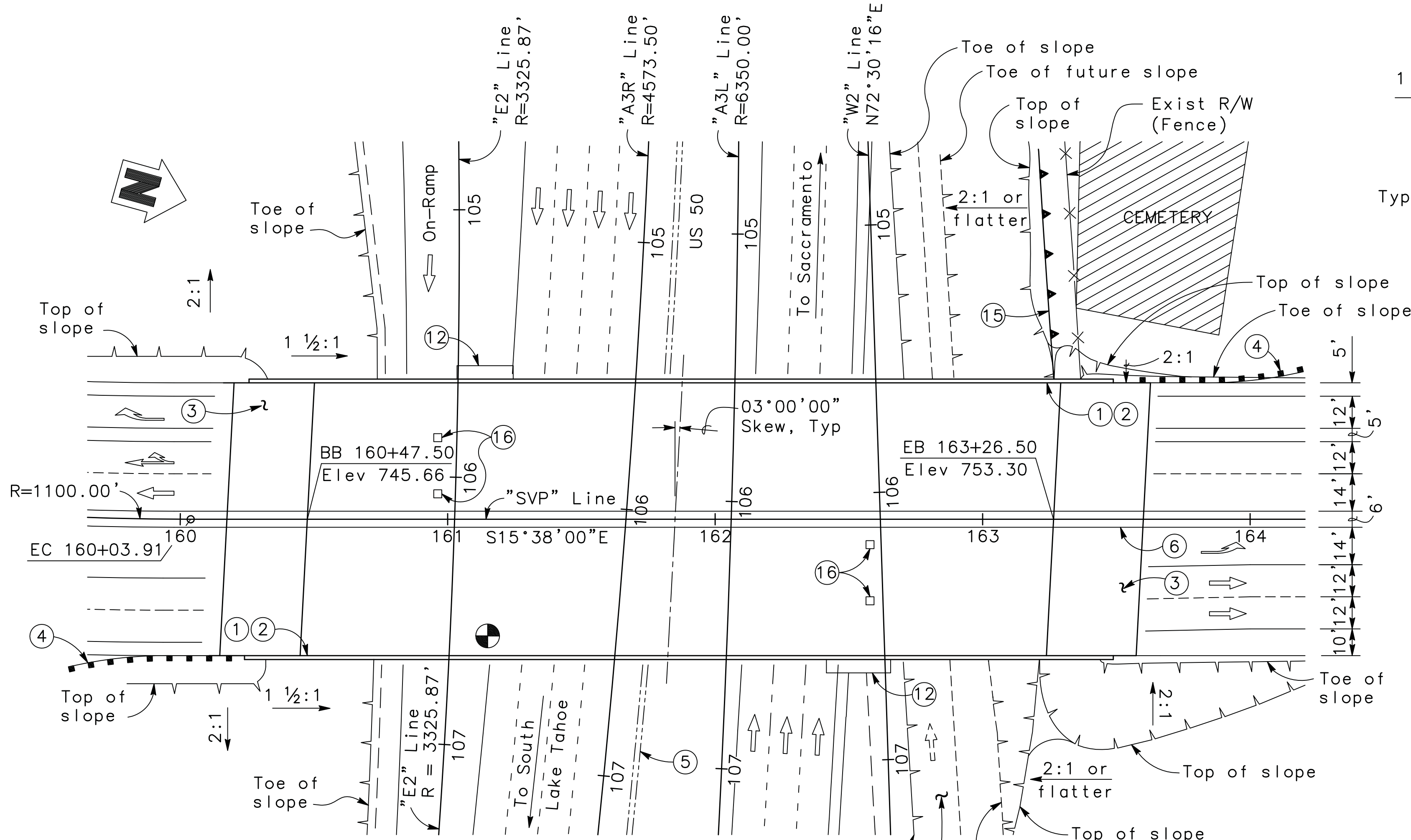
MARK THOMAS & COMPANY, INC.
7300 FOLSOM BOULEVARD, SUITE 203
SACRAMENTO, CA 95826



PROFILE GRADE
NO SCALE



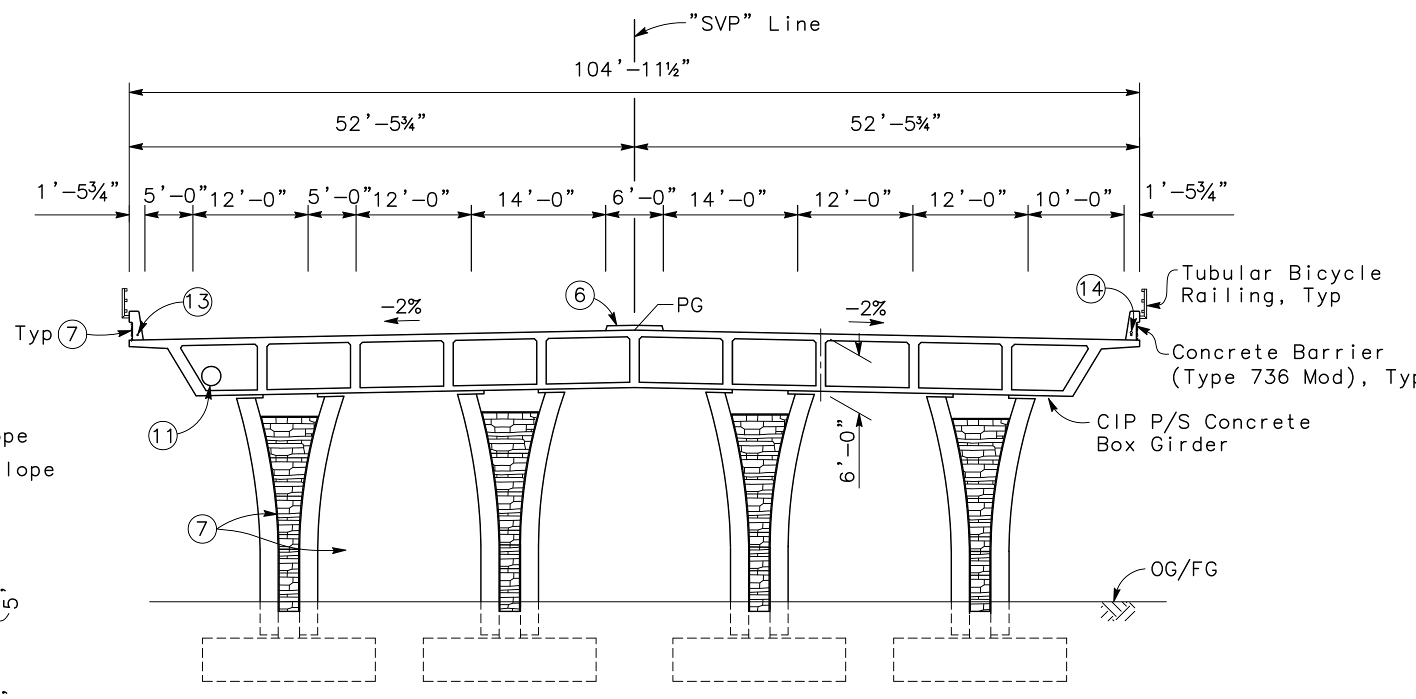
ELEVATION
1" = 30'



PLAN
1" = 30'

CURVE DATA

Control Line	R	Δ	T	L
"SVP" Line	1100.00'	50°56'31"	523.99'	978.02'
"E2" Line	3325.87'	08°41'21"	252.67'	504.38'
"A3R" Line	4573.50'	20°35'50"	831.03'	1644.12'
"A3L" Line	6350.00'	11°00'49"	612.20'	1220.63'



TYPICAL SECTION
1" = 10'

LEGEND

1. Paint "Silva Valley Parkway Overcrossing"
2. Paint "Br No.25-0127" and Year completed
3. Structure Approach Slab Type N(30S)
4. MBGR, see 'Road Plans'
5. Exist Concrete Barrier Type (60)
6. 8" Raised Median
7. Architectural Texture (Dry Stack Rock)
8. Slope Paving (FG - Phase 1)
9. Slope Paving (FG - Phase 2)
10. Grade Existing Slope (FG - Phase 1)
11. Future Utility Opening
12. Bridge Mounted Sign
13. 2" Dia conduit for electric, see 'Road Plans'
14. Two 2" conduits, see 'Road Plans'
15. Future Retaining Wall
16. Vehicle Loops, see 'Road Plans'

Indicates New Construction
Indicates Existing Structure
Indicates Point of Minimum Vertical Clearance

Notes:
1. For 'Index to Plans' & 'General Notes', see "Deck Contours" sheet.
2. For Foundation Information, see "Foundation Plan" sheet.
3. For Roadway Construction Phases, see 'Road Plans'.

DESIGN OVERSIGHT

SIGN OFF DATE

	BY	CHECKED			
DESIGN	P. PARK	S. MICHALSKI	LOAD & RESISTANCE FACTOR DESIGN	LIVE LOADING: HL93 PERMIT DESIGN VEHICLE	
DETAILS	T. TRAN	S. MICHALSKI	LAYOUT	BY D. MINNEMA	CHECKED P. PARK
QUANTITIES			SPECIFICATIONS	BY	PLANS AND SPECS COMPARED

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0127
POST MILES 1.8

SILVA VALLEY PARKWAY OVERCROSSING
GENERAL PLAN

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.06/2.90		

REGISTERED STRUCTURAL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER PO-KANG CHEN No. S3112 Exp. 9/30/13 STRUCTURAL STATE OF CALIFORNIA

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COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
4950 HILLSDALE CIR STE 400
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MARK THOMAS & CO., INC.
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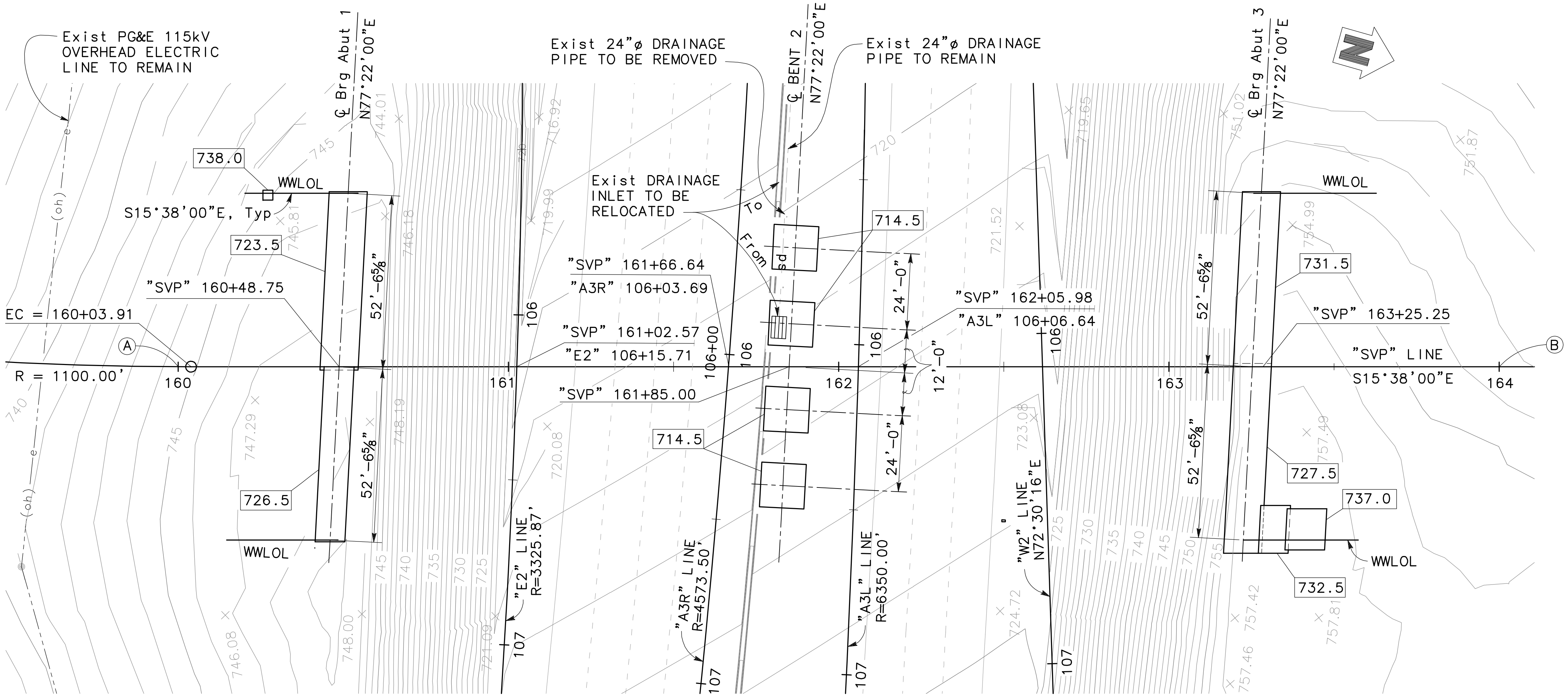


PLAN

1" = 20'

NOTE:

Utilities shown are for reference only. For above and below ground utilities, see 'UTILITY PLANS'.

SPREAD FOOTING DATA TABLE

Support Location	Working Stress Design (WSD)		Load and Resistance Factor Design (LRFD)		
	Permissible Gross Contact Stress (Settlement) (ksf)	Allowable Gross Bearing Capacity (ksf)	Service Permissible Net Contact Stress (settlement) (ksf)	Strength Factored Gross Normal Bearing Resistance φb = 0.45 (ksf)	Extreme Event Factored Gross Normal Bearing Resistance φb = 1.00 (ksf)
Abut 1	30	30	N/A	N/A	N/A
Bent 2	N/A	N/A	30	22.5	50
Abut 3	30	30	N/A	N/A	N/A

DATUM

National Geodetic Vertical Datum of 1988 (NGVD 29)
North American Datum of 1983 (NAD 83)

SVP LINE ALIGNMENT DATA

	Northing	Easting
(A) 160+00.00	1929656.429	6583956.914
(B) 164+00.00	1930100.132	6583961.713

BENCHMARKS

Benchmark No. 1:
Orthometric heights (elevation) shown are NGVD 29 based on HPGN D CA 03 DL having an elevation of 693.55 feet.

Benchmark No. 2:
USGS BM T 127 (PID JS0692) having an elevation of 673.08 feet.

LEGEND:

Indicates Bottom of Footing Elevation (feet)

757.81× Indicates spot elevation

11/08/10 APPROVAL DATE
GEOTECHNICAL PROFESSIONAL

SCALE: No Scale	VERT.DATUM NGVD 29	HORZ.DATUM CCS83(1991.35)Z2
PHOTOGRAMMETRY AS OF: 5/20/05	ALIGNMENT TIES	
SURVEYED BY M. Stringer	DRAFTED BY G. BOYKO	
FIELD CHECKED BY O. Senda	CHECKED BY	

DESIGN OVERSIGHT

SIGN OFF DATE

	BY	CHECKED
DESIGN	P. PARK	S. MICHALSKI
DETAILS	T. TRAN	S. MICHALSKI
QUANTITIES		

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0127
POST MILES 1.8

SILVA VALLEY PARKWAY OVERCROSSING

FOUNDATION PLAN

FOUNDATION PLAN SHEET (ENGLISH) (REV.7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 03-1E2901
PROJECT NUMBER & PHASE: 0300000258

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 07/28/10 11/08/10 03/23/12

SHEET 3 OF 24

FILE => $REQUEST

CONTRACT NO.:

PROJECT ID:

USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

APPENDIX C

Laboratory Test Results

LABORATORY TEST RESULTS

To classify the subsurface soil and obtain parameters for analysis, BCI performed laboratory tests on some of the samples obtained from the exploratory borings. Tests included:

- Moisture Content-Dry Density (ASTM D2937 & D2216)
- Unconfined Compressive Strength – Rock (ASTM 2938)

BCI performed laboratory tests in substantial conformance with the designated test procedure. The test results follow.

Laboratory Testing Summary

Exploration I.D.	Sample No.	Depth (feet)	Sample Type	USCS Classification	Unconfined Compression (psi)
R-10-001	Core	18.25-18.9	HQ	Rock	32,330
R-10-002	Core	5.4-6.5	HQ	Rock	23,000
R-10-003	Core	20.0-20.7	HQ	Rock	15,750

APPENDIX D

Calculations and Analyses

Silva Valley Parkway

Comparison of ARS Curves
(unlock sheet with "shmi")

Model Inputs

Fault

Magnitude	6.5	(5 to 8.5)
F_{RV}	0	(input 1 = Rev)
F_{NM}	1	(input 1 = Normal)
Dip (degree)	90	(0 to 90)
Z_{TOR} (km)	0	

Distance

R_{RUP} (km)	12.6	
R_{JB} (km)	12.6	
R_x (km)	12.5	
Hanging Wall?	☐ Yes?	
Near-Field Factor?	☑ Yes?	

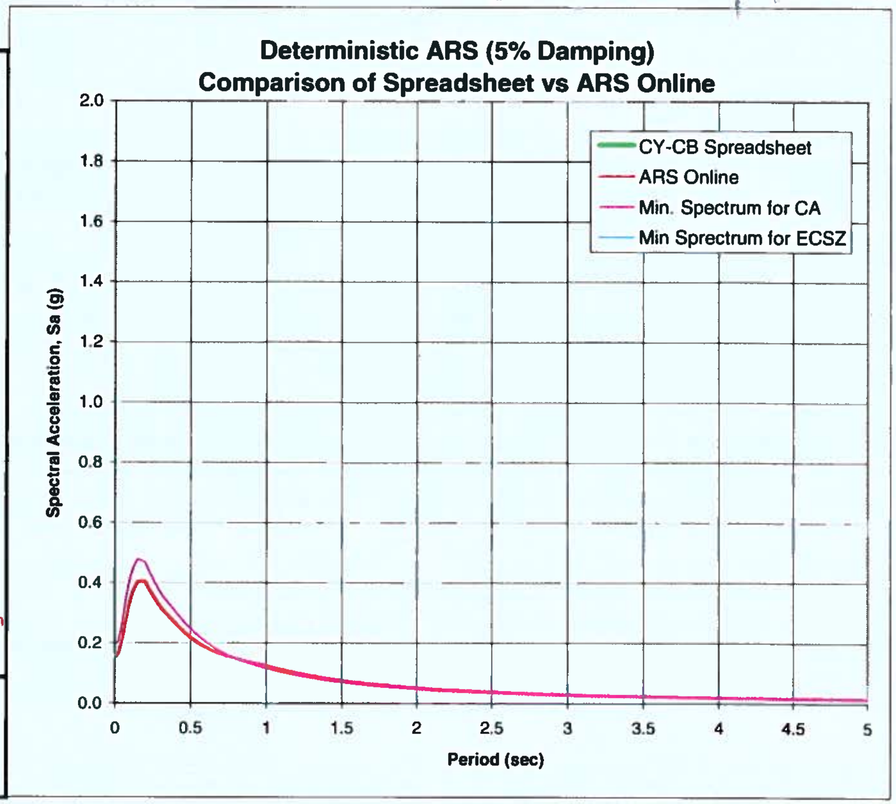
Site

V_{S30} (m/sec)	760	(270 to 1500 m/s)
$Z_{1.0}$ (m)	0	(0 - No Basin)
$Z_{2.5}$ (km)	0	(0 - No Basin)
No. Cal. Basin?	☐ Yes ?	(Check only for sites located within a Basin)
So. Cal. Basin?	☐ Yes ?	

Analysis

ARS Online vs CY-CB Spreadsheet Results

MAX. % Diff. = 2%



Page 1 of 3

SVP

CY-CB Spreadsheet Results	
T (sec)	CB-CY S(a)
0.010	0.15777
0.020	0.16069
0.022	0.16319
0.025	0.16687
0.029	0.17168
0.030	0.17300
0.032	0.17701
0.035	0.18307
0.036	0.18515
0.040	0.19326
0.042	0.19744
0.044	0.20160
0.045	0.20372
0.046	0.20583
0.048	0.20999
0.050	0.21418
0.055	0.22787
0.060	0.24151
0.065	0.25480
0.067	0.26013
0.070	0.26782
0.075	0.28042
0.080	0.29266
0.085	0.30463
0.090	0.31605
0.095	0.32727
0.100	0.33793
0.110	0.35410
0.120	0.36863
0.130	0.38134
0.133	0.38460
0.140	0.39198
0.150	0.40130
0.160	0.40297
0.170	0.40356
0.180	0.40360
0.190	0.40302
0.200	0.40211
0.220	0.38341
0.240	0.36635
0.250	0.35832
0.260	0.34988
0.280	0.33451
0.290	0.32714
0.300	0.32013
0.320	0.30761
0.340	0.29585
0.350	0.29032
0.360	0.28495
0.380	0.27479
0.400	0.26534
0.420	0.25437
0.440	0.24410
0.450	0.23933
0.460	0.23468
0.480	0.22584

Place ARS Online Deterministic Data Here "Paste"					
T (sec)	Base S(a)	Basin Factor	Near Fault Factor	Final Adj. S(a)	Diff. (%)
0.01	0.158	1	1	0.158	0%
0.02	0.161	1	1	0.161	0%
0.022	0.163	1	1	0.163	0%
0.025	0.167	1	1	0.167	0%
0.029	0.171	1	1	0.171	0%
0.03	0.173	1	1	0.173	0%
0.032	0.177	1	1	0.177	0%
0.035	0.183	1	1	0.183	0%
0.036	0.185	1	1	0.185	0%
0.04	0.193	1	1	0.193	0%
0.042	0.197	1	1	0.197	0%
0.044	0.201	1	1	0.201	0%
0.045	0.203	1	1	0.203	0%
0.046	0.206	1	1	0.206	0%
0.048	0.21	1	1	0.21	0%
0.05	0.214	1	1	0.214	0%
0.055	0.228	1	1	0.228	0%
0.06	0.241	1	1	0.241	0%
0.065	0.255	1	1	0.255	0%
0.067	0.26	1	1	0.26	0%
0.07	0.268	1	1	0.268	0%
0.075	0.28	1	1	0.28	0%
0.08	0.292	1	1	0.292	0%
0.085	0.304	1	1	0.304	0%
0.09	0.316	1	1	0.316	0%
0.095	0.327	1	1	0.327	0%
0.1	0.338	1	1	0.338	0%
0.11	0.354	1	1	0.354	0%
0.12	0.368	1	1	0.368	0%
0.13	0.381	1	1	0.381	0%
0.133	0.384	1	1	0.384	0%
0.14	0.392	1	1	0.392	0%
0.15	0.401	1	1	0.401	0%
0.16	0.403	1	1	0.403	0%
0.17	0.403	1	1	0.403	0%
0.18	0.403	1	1	0.403	0%
0.19	0.403	1	1	0.403	0%
0.2	0.402	1	1	0.402	0%
0.22	0.383	1	1	0.383	0%
0.24	0.366	1	1	0.366	0%
0.25	0.358	1	1	0.358	0%
0.26	0.349	1	1	0.349	0%
0.28	0.334	1	1	0.334	0%
0.29	0.327	1	1	0.327	0%
0.3	0.32	1	1	0.32	0%
0.32	0.307	1	1	0.307	0%
0.34	0.296	1	1	0.296	0%
0.35	0.29	1	1	0.29	0%
0.36	0.285	1	1	0.285	0%
0.38	0.274	1	1	0.274	0%
0.4	0.265	1	1	0.265	0%
0.42	0.254	1	1	0.254	0%
0.44	0.244	1	1	0.244	0%
0.45	0.239	1	1	0.239	0%
0.46	0.234	1	1	0.234	0%
0.48	0.226	1	1	0.226	0%

For Comparsion Plots of Min. Sprectra, Paste Special into Cells			
Min. Spectrum for CA		Min Sprectrum for ECSZ	
T (sec)	S (a)	T (sec)	S (a)
0.01	0.197		
0.02	0.201		
0.022	0.204		
0.025	0.208		
0.029	0.214		
0.03	0.216		
0.032	0.221		
0.035	0.228		
0.036	0.231		
0.04	0.241		
0.042	0.246		
0.044	0.251		
0.045	0.254		
0.046	0.256		
0.048	0.262		
0.05	0.267		
0.055	0.284		
0.06	0.3		
0.065	0.317		
0.067	0.323		
0.07	0.333		
0.075	0.348		
0.08	0.362		
0.085	0.376		
0.09	0.389		
0.095	0.401		
0.1	0.414		
0.11	0.43		
0.12	0.445		
0.13	0.458		
0.133	0.461		
0.14	0.468		
0.15	0.476		
0.16	0.476		
0.17	0.474		
0.18	0.472		
0.19	0.469		
0.2	0.466		
0.22	0.444		
0.24	0.423		
0.25	0.413		
0.26	0.403		
0.28	0.386		
0.29	0.377		
0.3	0.369		
0.32	0.354		
0.34	0.34		
0.35	0.333		
0.36	0.327		
0.38	0.315		
0.4	0.303		
0.42	0.291		
0.44	0.279		
0.45	0.273		
0.46	0.267		
0.48	0.257		

Analysis of CY-CB Attenuation Prediction Equation vs ARS Online Results

SVP

0.500	0.21774
0.550	0.20041
0.600	0.18607
0.650	0.17400
0.660	0.17114
0.700	0.16373
0.750	0.15484
0.800	0.14725
0.850	0.14054
0.900	0.13446
0.950	0.12902
1.000	0.12403
1.100	0.11078
1.200	0.09980
1.300	0.09043
1.400	0.08237
1.500	0.07537
1.600	0.06924
1.700	0.06389
1.800	0.05914
1.900	0.05492
2.000	0.05121
2.200	0.04500
2.400	0.03999
2.500	0.03784
2.600	0.03588
2.800	0.03245
3.000	0.02954
3.200	0.02715
3.400	0.02508
3.500	0.02413
3.600	0.02325
3.800	0.02164
4.000	0.02021
4.200	0.01900
4.400	0.01792
4.600	0.01694
4.800	0.01604
5.000	0.01522

0.5	0.217	1	1	0.217	0%
0.55	0.196	1	1.02	0.2	0%
0.6	0.179	1	1.04	0.186	0%
0.65	0.164	1	1.06	0.174	0%
0.667	0.159	1	1.067	0.17	1%
0.7	0.151	1	1.08	0.164	0%
0.75	0.141	1	1.1	0.155	0%
0.8	0.131	1	1.12	0.147	0%
0.85	0.123	1	1.14	0.14	0%
0.9	0.116	1	1.16	0.134	0%
0.95	0.109	1	1.18	0.129	0%
1	0.103	1	1.2	0.124	0%
1.1	0.092	1	1.2	0.111	0%
1.2	0.083	1	1.2	0.1	0%
1.3	0.075	1	1.2	0.09	0%
1.4	0.069	1	1.2	0.082	0%
1.5	0.063	1	1.2	0.075	0%
1.6	0.058	1	1.2	0.069	0%
1.7	0.053	1	1.2	0.064	0%
1.8	0.049	1	1.2	0.059	0%
1.9	0.046	1	1.2	0.055	0%
2	0.043	1	1.2	0.051	0%
2.2	0.038	1	1.2	0.045	0%
2.4	0.033	1	1.2	0.04	0%
2.5	0.032	1	1.2	0.038	0%
2.6	0.03	1	1.2	0.036	0%
2.8	0.027	1	1.2	0.033	2%
3	0.025	1	1.2	0.03	2%
3.2	0.023	1	1.2	0.027	1%
3.4	0.021	1	1.2	0.025	0%
3.5	0.02	1	1.2	0.024	1%
3.6	0.019	1	1.2	0.023	1%
3.8	0.018	1	1.2	0.022	2%
4	0.017	1	1.2	0.02	1%
4.2	0.016	1	1.2	0.019	0%
4.4	0.015	1	1.2	0.018	0%
4.6	0.014	1	1.2	0.017	0%
4.8	0.013	1	1.2	0.016	0%
5	0.013	1	1.2	0.015	1%

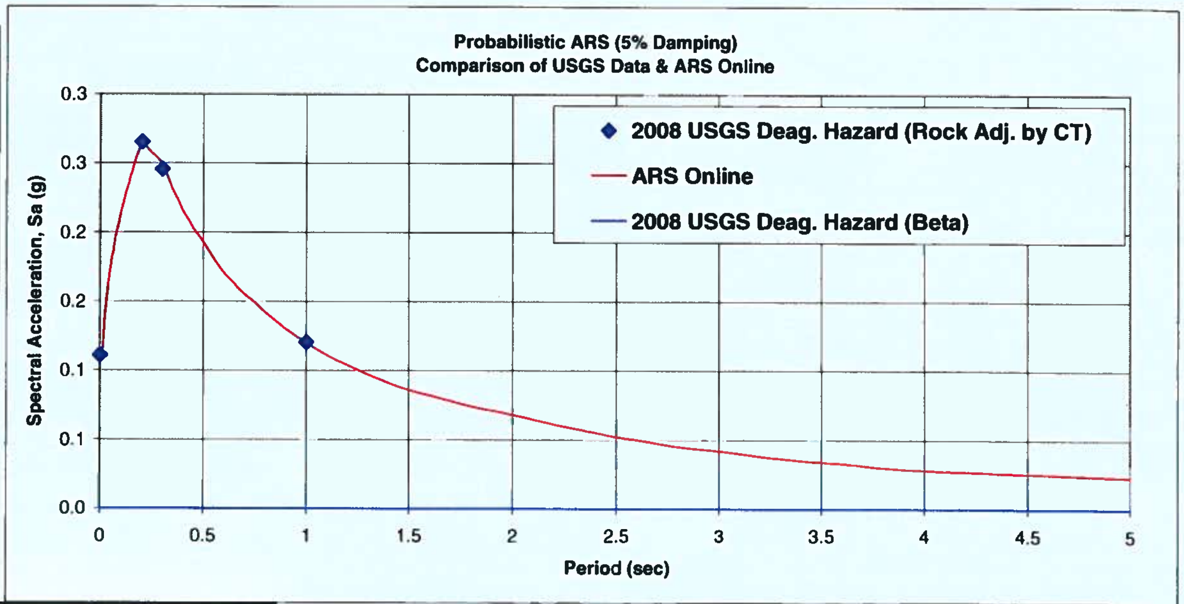
0.5	0.248		
0.55	0.223		
0.6	0.203		
0.65	0.185		
0.667	0.18		
0.7	0.171		
0.75	0.158		
0.8	0.148		
0.85	0.138		
0.9	0.13		
0.95	0.122		
1	0.115		
1.1	0.103		
1.2	0.093		
1.3	0.084		
1.4	0.076		
1.5	0.07		
1.6	0.064		
1.7	0.059		
1.8	0.054		
1.9	0.051		
2	0.047		
2.2	0.041		
2.4	0.037		
2.5	0.035		
2.6	0.033		
2.8	0.03		
3	0.027		
3.2	0.025		
3.4	0.023		
3.5	0.022		
3.6	0.021		
3.8	0.02		
4	0.018		
4.2	0.017		
4.4	0.016		
4.6	0.015		
4.8	0.015		
5	0.014		

Silva Valley Parkway

Comparison spreadsheet of the 2008 USGS Probabilistic Seismic Hazard Data and ARS Online Probabilistic Data
Spectral Accelerations Points from USGS Website at http://earthquake.usgs.gov/research/hazmaps/products_data/2008/data/ (unlock spreadsheet "shmi")

*** Note: This spreadsheet uses the given latitude and longitude data provided by the user to estimate spectral acceleration values with a probability of exceedence 5% in 50 yrs (or 975 yr return period). The four spectral acceleration data points plotted on the graph are from the USGS website and are based on a 0.05 degree grid. Basic interpolation is used to estimate intermediate values inside each grid. Raw Data points are provided in the tabs of this spreadsheet. Corner grid spectral acceleration data are shown in the "calculation" tab.**

Input Site Information	
Latitude	Longitude
38.6581	-121.0543
V_{S30} (m/s) =	760
Near Fault Factor, Derived from USGS Deagg. Dist (km) =	123
$Z_{1.0}$ (m) =	0
$Z_{2.5}$ (km) =	0



Place ARS Online Probabilistic Data Here "Paste"

T (sec)	Base Spectrum S(a)	Basin Factor	Near Fault Factor	Final Adj. Spectrum S(a)
0.01	0.111	1	1	0.111
0.02	0.135	1	1	0.135
0.022	0.139	1	1	0.139
0.025	0.144	1	1	0.144
0.029	0.151	1	1	0.151
0.03	0.152	1	1	0.152
0.032	0.155	1	1	0.155
0.035	0.159	1	1	0.159
0.036	0.16	1	1	0.16
0.04	0.165	1	1	0.165
0.042	0.167	1	1	0.167
0.044	0.17	1	1	0.17
0.045	0.171	1	1	0.171
0.046	0.172	1	1	0.172
0.048	0.174	1	1	0.174
0.05	0.176	1	1	0.176
0.055	0.181	1	1	0.181
0.06	0.185	1	1	0.185
0.065	0.19	1	1	0.19
0.067	0.191	1	1	0.191
0.07	0.194	1	1	0.194
0.075	0.198	1	1	0.198
0.08	0.201	1	1	0.201
0.085	0.205	1	1	0.205
0.09	0.208	1	1	0.208
0.095	0.211	1	1	0.211
0.1	0.215	1	1	0.215
0.11	0.221	1	1	0.221
0.12	0.227	1	1	0.227
0.13	0.233	1	1	0.233
0.133	0.234	1	1	0.234
0.14	0.238	1	1	0.238
0.15	0.243	1	1	0.243
0.16	0.248	1	1	0.248
0.17	0.253	1	1	0.253
0.18	0.257	1	1	0.257
0.19	0.262	1	1	0.262
0.2	0.266	1	1	0.266
0.22	0.262	1	1	0.262
0.24	0.259	1	1	0.259
0.25	0.257	1	1	0.257

Analysis of ARS Online Results vs USGS Deaggregation Hazard (Adj. By CT)

Period (sec)	USGS Interpolated Spectral Accel.	Adj. for Near Fault Effect	Adj. for Soil Amplification	Adj. For Basin Effect	Final Adj. USGS Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0	0.110	1.000	1.007	1.000	0.111	0.111	0.1%
0.2	0.265	1.000	1.003	1.000	0.265	0.266	-0.3%
0.3	0.244	1.000	1.006	1.000	0.246	0.25	-1.7%
1	0.121	1.000	1.000	1.000	0.121	0.12	0.4%

Max % Difference = 1.7%

USGS Deaggregation Hazard (Beta) with Near Field and Basin Factors

Period (sec)	INPUT USGS Deagg. Spec Accel	Adj. for Near Fault Effect	Adj. For Basin Effect	Final Adj. USGS Deagg Spec Accel	ARS Online Final Adj. Spect. Accel.	% Difference (bet. USGS & ARS Online)
0		1.000	1.000		0.111	
0.1		1.000	1.000		0.215	
0.2		1.000	1.000		0.266	
0.3		1.000	1.000		0.25	
0.5		1.000	1.000		0.193	0.0%
1		1.000	1.000		0.12	0.0%
2		1.000	1.000		0.068	0.0%
3		1.000	1.000		0.042	0.0%
4		1.000	1.000		0.028	0.0%
5		1.000	1.000		0.023	0.0%

Max % Difference = 0.0%

Page 1 of 1

blackburn consulting

Project Silva OC	Client MTCo		Page No. 1 of 3
Subject Bearing Capacity	File # 556	By PFF	Date 10/29/10 rev-3/12

Bearing Capacity on Rock

For abutments, footings will be at an elevation with RQD greater than 40%, thin to tight joints, and moderate weathering.

From BDS Figure 4.4.8.1.1A
q_{all} = approx 40 tsf

Check using equation 4.4.8.1.2-1

(see page 2 & 3 of 3)

$q_{ult} = N_{ms} C_o$ $N_{ms} = 0.05$ $C_o \sim 2{,}000$ ksf

$= 0.05\,(2{,}000\ \text{ksf}) = 100\ \text{ksf} = \underline{50\ \text{tsf}}$

$q_{all} = 50\ \text{tsf}/3 = 17\ \text{tsf} = \underline{34\ \text{ksf}}$

use $\underline{30\ \text{ksf}}$ allowable

For WSD

q_{pg} = 40 tsf plus for 1" of settlement

q_{all} = 34 to 40 tsf → use 30 tsf

LRFD

q_{pn} = 40 tsf plus

Strength $q_R = 0.45\,(50\ \text{tsf}) = 22.5\ \text{tsf}$

Extreme $q_R = 1.0\,(50\ \text{tsf}) = 50\ \text{tsf}$

Check: DOS

or open not wider than 1/8 inch. For footings on less competent rock, more detailed investigations and analyses should be used to account for the effects of weathering, the presence and condition of discontinuities, and other geologic factors.

4.4.8.1 Bearing Capacity

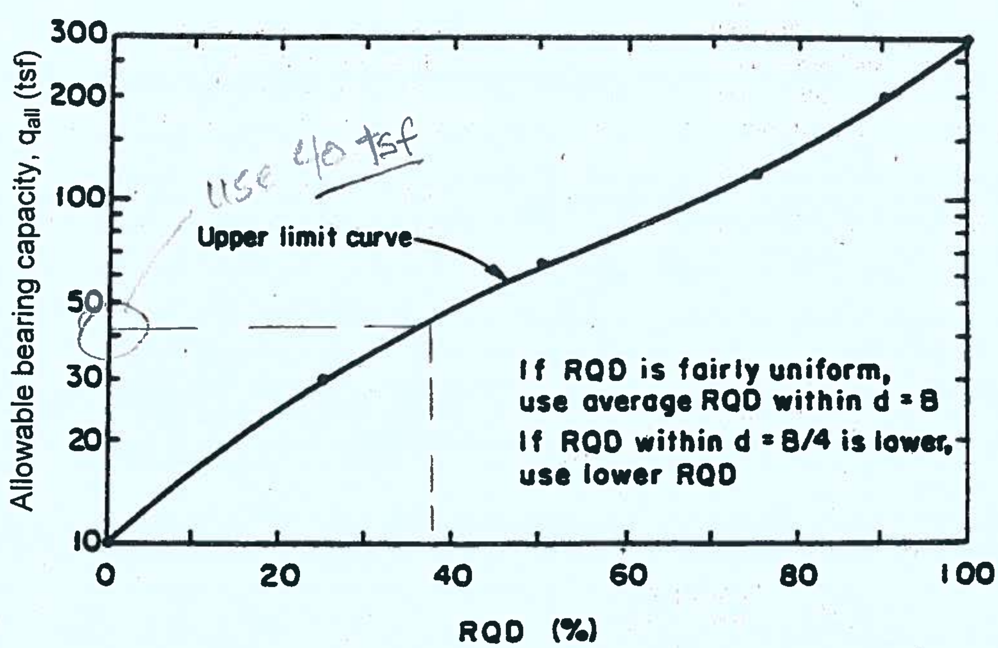
4.4.8.1.1 Footings on Competent Rock

The allowable bearing capacity for footings supported on level surfaces in competent rock may be determined using Figure 4.4.8.1.1 A (Peck, et al. 1974). In no instance shall the maximum allowable bearing capacity exceed the allowable bearing stress in the concrete. The RQD used in Figure 4.4.8.1.1A shall be the average RQD for the rock within a depth of B below the base of the footing, where the RQD values are relatively uniform within that interval. If rock within a depth of 0.5B below the base of the footing is of poorer quality, the RQD of the poorer rock shall be used to determine q_{all}.

4.4.8.1.2 Footings on Broken or Jointed Rock

The design of footings on broken or jointed rock must account for the condition and spacing of joints and other discontinuities. The ultimate bearing capacity of footings on broken or jointed rock may be estimated using the following relationship:

$$q_{ult} = N_{ms}C_o \qquad (4.4.8.1.2\text{-}1)$$



Note:
q_{all} shall not exceed the unconfined compressive strength of the rock or 0.595 f'_c of the concrete.

FIGURE 4.4.8.1.1A Allowable Contact Stress for Footings on Rock with Tight Discontinuities Peck, et al. (1974)

Pg 2 of 3

TABLE 4.4.8.1.2A Values of Coefficient N_{ms} for Estimation of the Ultimate Bearing Capacity of Footings on Broken or Jointed Rock (Modified after Hoek, (1983))

Rock Mass Quality	General Description	RMR(1) Rating	NGI(2) Rating	RQD(3) (%)	N_{ms}(4) A	B	C	D	E
Excellent	Intact rock with joints spaced > 10 feet apart	100	500	95-100	3.8	4.3	5.0	5.2	6.1
Very good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 3 to 10 feet apart	85	100	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 feet apart	65	10	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 1 to 3 feet apart	44	1	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced I to 20 inches apart with some gouge	23	0.1	25-50	0.015	0.016	0.019	0.020	0.024
Very poor	Rock with numerous highly weathered joints spaced < 2 inches apart	3	0.01	< 25	Use q_{ult} for an equivalent soil mass				

Foundation rock falls into this range

Use mid value of 0.05 = Nms

(1)Geomechanics Rock Mass Rating (RMQ) System–Bieniawski, 1988.
(2)Norwegian Geotechnical Institute (NGI) Rock Mass Classification System, Barton, et al., 1974.
(3)Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR or NGI rating systems.
(4)Value of N_{ms} as a function of rock type; refer to Table 4.4.8.1.2B for typical range of values of C_o for different rock type in each category.

Refer to Table 4.4.8.1.2A for values of N_{ms}. Values of C_o should preferably be determined from the results of laboratory testing of rock cores obtained within 2B of the base of the footing. Where rock strata within this interval are variable in strength, the rock with the lowest capacity should be used to determine q_{ult}. Alternatively, Table 4.4.8.1.2B may be used as a guide to estimate C_o. For rocks defined by very poor quality, the value of q_{ult} should be determined as the value of q_{ult} for an equivalent soil mass.

4.4.8.1.3 Factors of Safety

Spread footings on rock shall be designed for Group 1 loadings using a minimum factor of safety (FS) of 3.0 against a bearing capacity failure.

4.4.8.2 Settlement

4.4.8.2.1 Footings on Competent Rock

For footings on competent rock, elastic settlements will generally be less than 1/2 inch when footings are designed in accordance with Article 4.4.8.1.1. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics must be made. For rock masses which have time-dependent settlement characteristics, the procedure in Article 4.4.7.2.3 may be followed to determine the time-dependent component of settlement.

EQUIVALENT FLUID WEIGHTS (EFWs)

Project: Silva Valley Parkway OC
BCI No.: 556.2
Date: 11/4/2010
By: PFF

Lateral Pressures
Pg 1 of 1

EFWs for static condition determined using equations in: Naval Facilities (NAVFAC) Design Manual 7.2 for active (K_A) and passive (K_P) lateral coefficients; and USACE Retaining and Floodwalls Manual (EM 1110-2-2502) for at-rest (K_O) lateral coefficient.
EFWs for seismic loading conditions determined using the Mononobe-Okabe equation for active and passive lateral coefficients K_{AE} and K_{PE}.

Unit wieght of soil (pcf),	γ =	120.0	
Internal friction angle of soil (degrees),	ϕ =	33.0	(<45°)
Inclination of wall with respect to vertical (degrees),	β =	0.0	
Wall friction angle (degrees),	δ =	22.0	($\delta = 2\phi/3$)
Inclination of soil surface above wall (degrees),	i =	0.0	
Peak Ground Acceleration (g),	PGA =	0.20	
Horizontal seismic acceleration coefficient,	k_h =	0.10	
Vertical seismic acceleration coefficient,	k_v =	0.00	
Lateral wall displacement (inches),	d =	1.00	($1 \le d \ge 8$)

EFW = Kγ

EFW	Factor of Safety 1.0	1.5	2.0	
Active	36	--	--	psf/f
* Passive	407	271	203	psf/f
At rest	55	--	--	psf/f
Active_{EQ}	40	--	--	psf/f
* Passive_{EQ}	384	256	192	psf/f
At rest_{EQ}	62	--	--	psf/f

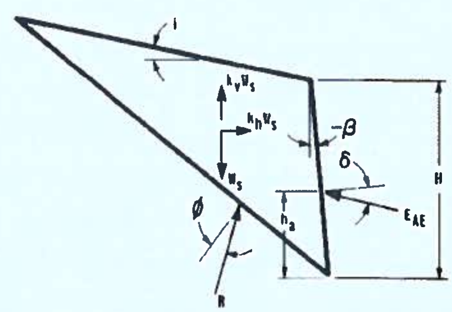
K_A =	0.29
K_P =	3.39
K_O =	0.46
K_{AE} =	0.33
K_{PE} =	3.20

Coefficient of Friciton (sliding) = tan(0.75ϕ) = 0.46

Static Loading

Active Pressure Coefficient (K_A):

$$K_A = [\cos\phi/\{1 + [\sin\phi(\sin\phi - \cos\phi\tan i)]^{0.5}\}]^2$$



Passive Pressure Coefficient (K_P):

$$K_P = [\cos\phi/\{1 - [\sin\phi(\sin\phi + \cos\phi\tan i)]^{0.5}\}]^2$$

At-rest Pressure Coefficient (K_O):

$$K_O = (1 - \sin\phi) \cdot (1 + \sin i)$$

Seismic Loading

Seismic Active Pressure Coefficient (K_{AE}):

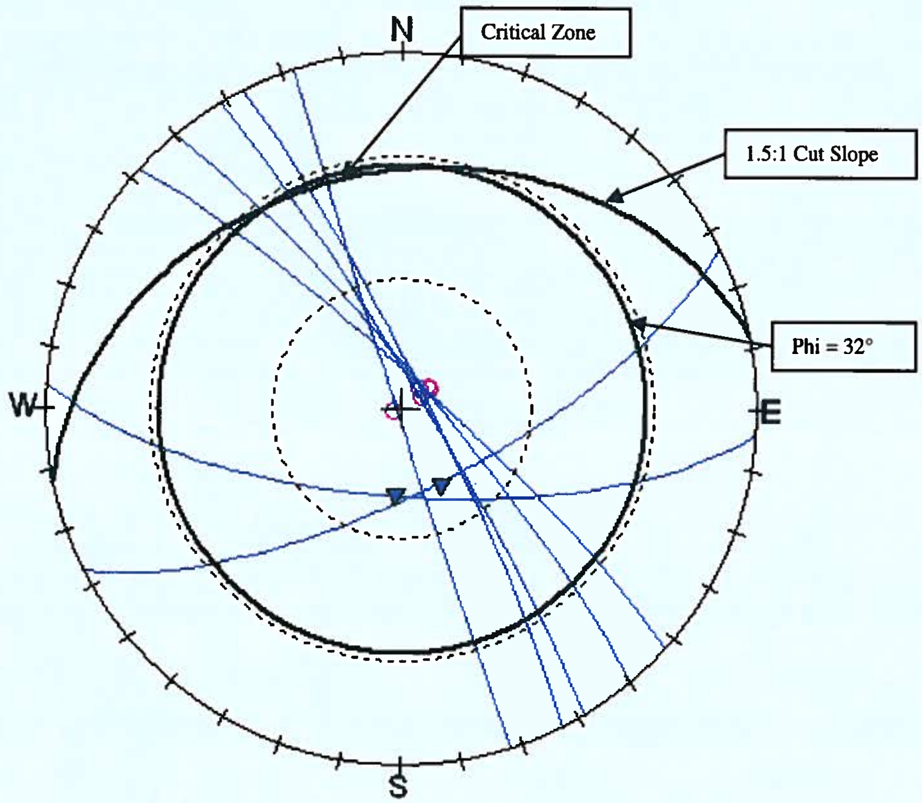
$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta\cos^2\beta\cos(\delta + \beta + \theta)} \times \left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^{-2}$$

Seismic Passive Pressure Coefficient (K_{PE}):

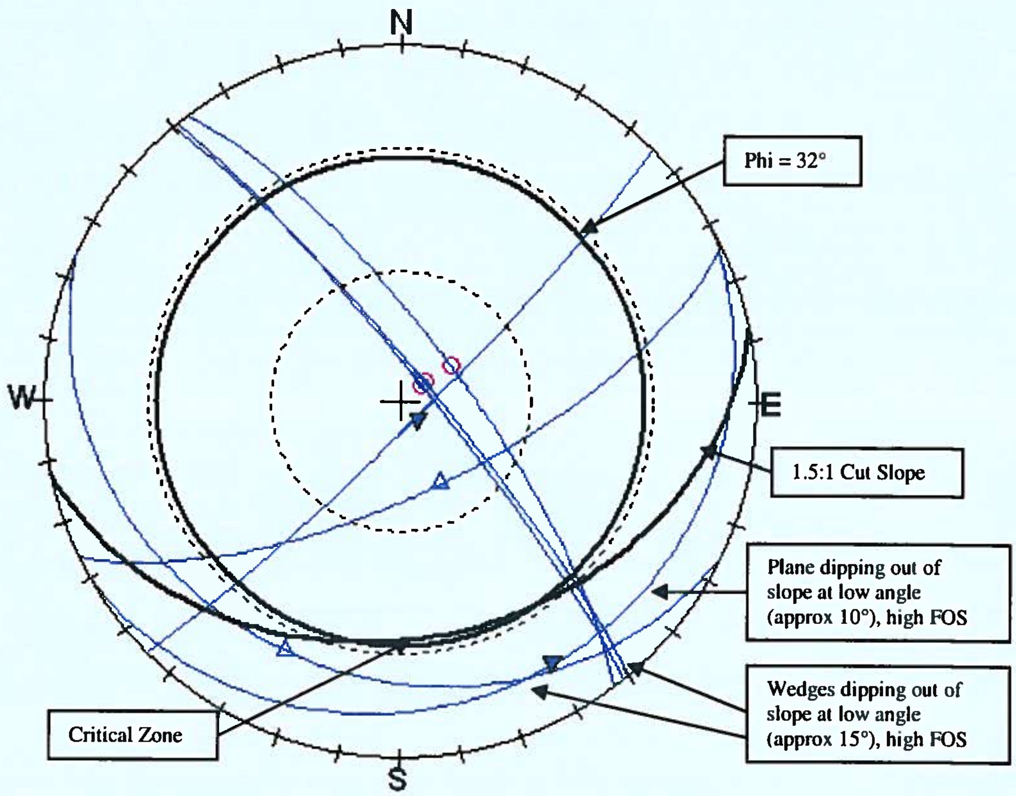
$$K_{PE} = \frac{\cos^2(\phi - \theta + \beta)}{\cos\theta\cos^2\beta\cos(\delta - \beta + \theta)} \times \left[1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta + i)}{\cos(\delta - \beta + \theta)\cos(i - \beta)}}\right]^{2}$$

1) For Seismic Active Case: $\phi \ge \theta + i$
2) For Seismic Passive Case: $\phi \ge \theta - i$
3) $k_h \approx 0.74A(A/d)^{0.25}$; A = PGA (Section 11.6.5, AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007)
4) For $k_h \le 0.2$, neglect k_v
5) For $k_h \ge 0.2$, $k_v \approx k_h/2$
6) Seismic Passive case
   neglects wall friction

* Level Backfill Condition Only.



Stereonet Plot of Discontinuities, Silva Valley Parkway OC, 1.5:1 Cut At South Abutment (approximately A3R Line, 104+00 to 108+00)



Stereonet Plot of Discontinuities, Silva Valley Parkway OC, 1.5:1 Cut At North Abutment (approximately A3L Line, 103+00 to 108+00)

Comprehensive Wedge Failure Analysis Input Data

Silva OC – North Cut

Abut 3

Wedge - Plunge ~16°

FOS > 5.0

(GR) Density of Rock = 170 lb(f)/ft ^{3}
(H1) Slope Height Referred to Plane 1 = 10 ft
(TL) Distance of Tension Crack from Crest along Plane 1 Trace = 25 ft

Plane 1: (D1) Dip Value = 25 °
(E1) Dip Direction = 205 °
Plane 2: (D1) Dip Value = 16 °
(E1) Dip Direction = 150 °
Plane 3: (D1) Dip Value = 0 °
(E1) Dip Direction = 0 °
Plane 4: (D1) Dip Value = 33 °
(E1) Dip Direction = 175 °
Plane 5: (D5) Dip Value = 33 °
(E5) Dip Direction = 175 °

Plane 1 : (C1) Cohesion = 250 lb(f)/ft ^{2}
(P1) Friction Angle = 32 °
Plane 2 : (C2) Cohesion = 250 lb(f)/ft ^{2}
(P2) Friction Angle = 32 °

Water Pressure : Dry Slope

The slope face DOES NOT hang over the toe of the slope.

Comprehensive Wedge Failure Analysis Output Data

(F) Factor of Safety = 5.27
CONTACT MAINTAINED ON BOTH PLANES

Intersection of Planes 1 & 2: Plunge = 15.98 °
Trend = 152.89 °

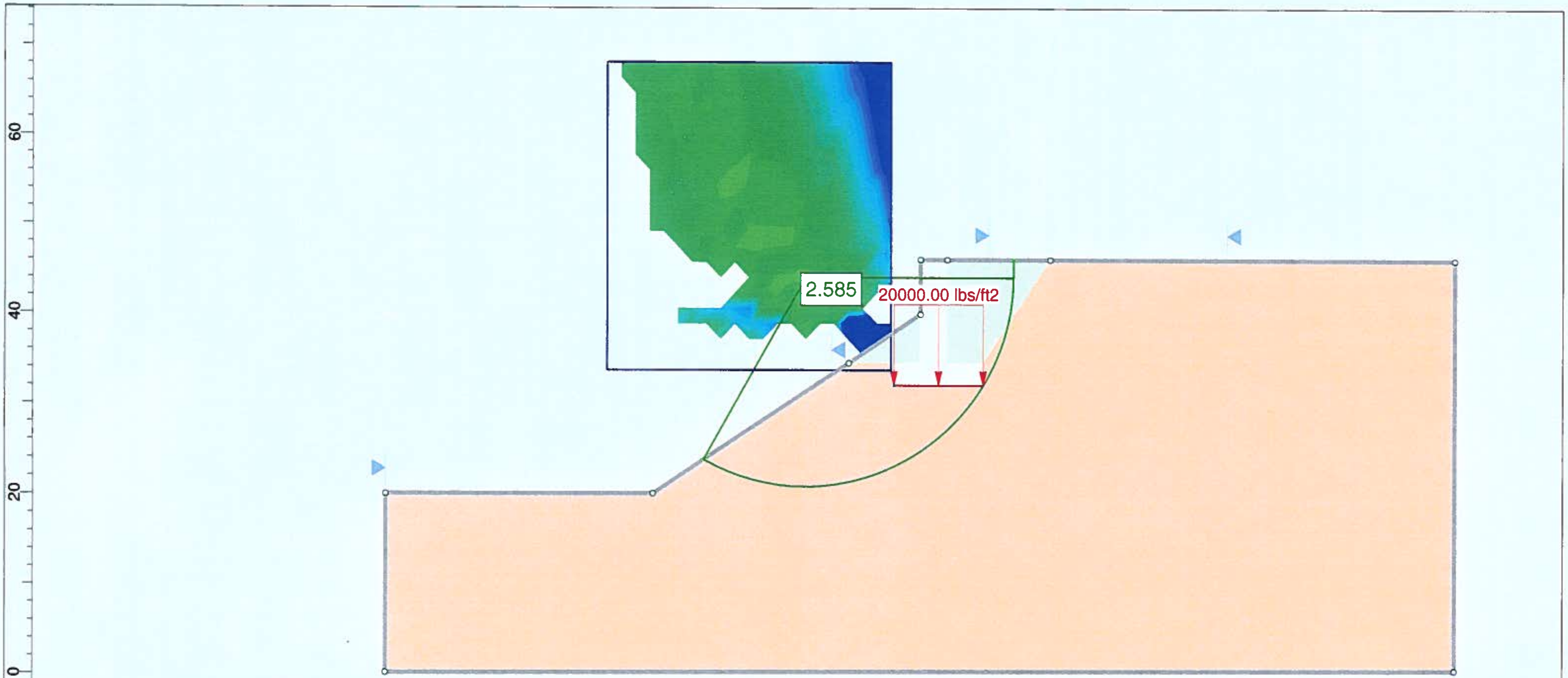
Weight of Wedge = 309920.33 lb(f)
Volume of Wedge = 1823.06 ft ^{3}

(N1) Effective Normal Force on Plane 1 = 11945.83 lb(f)
(N2) Effective Normal Force on Plane 2 = 286709.18 lb(f)

(A1) Plane 1 Area = 373.45 ft ^{2}
(A2) Plane 2 Area = 677.44 ft ^{2}

(H5) Tension Crack Height Along Planes 1 & 5 Intersection = 22.82 ft
(A5) Tension Crack Face Area = 41.61 ft ^{2}

Resisting Forces = 449342.92 lb(f)
Driving Forces = 85325.21 lb(f)

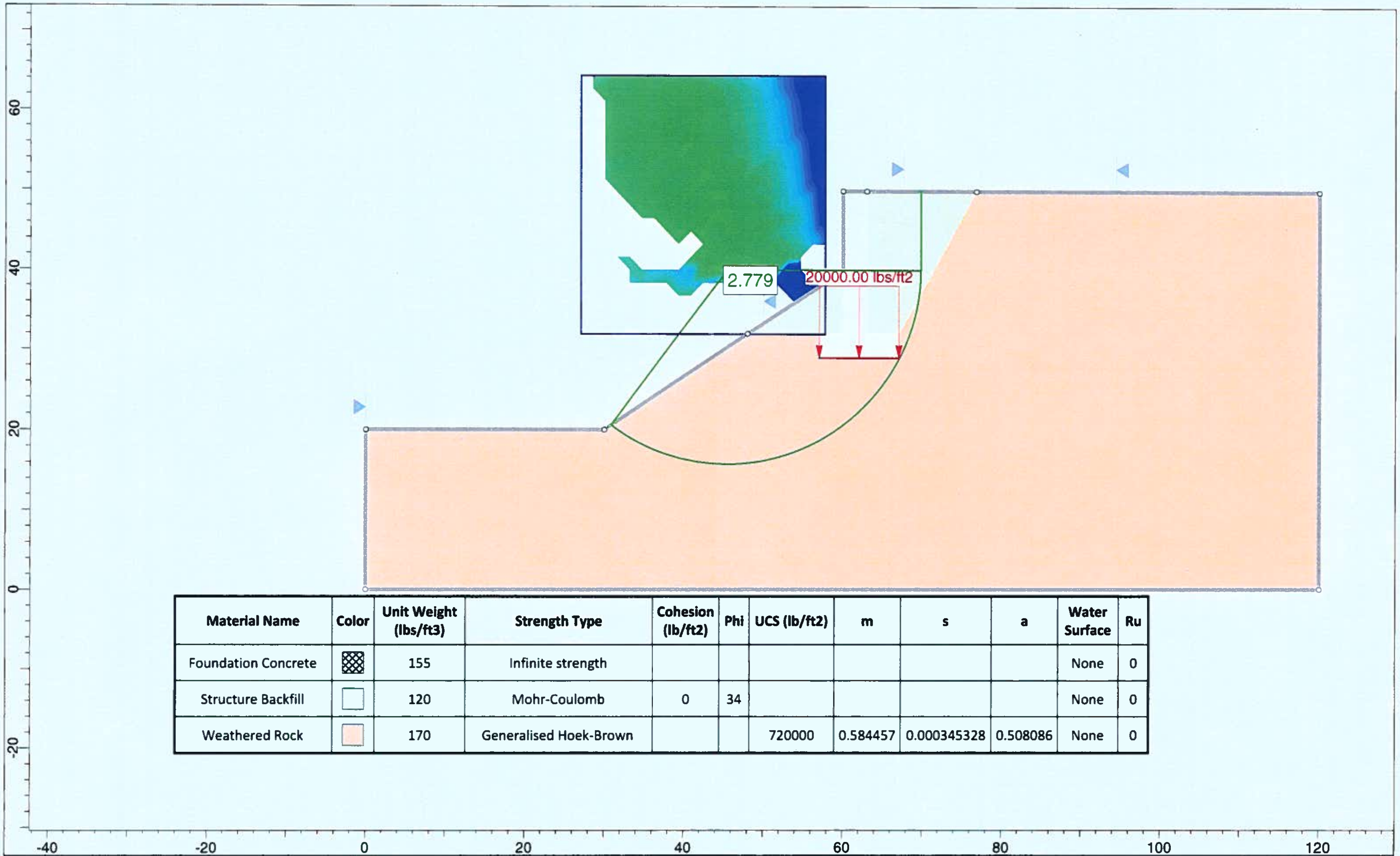


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	UCS (lb/ft2)	m	s	a	Water Surface	Ru
Foundation Concrete		155	Infinite strength							None	0
Structure Backfill		120	Mohr-Coulomb	0	34					None	0
Weathered Rock		170	Generalised Hoek-Brown			720000	0.584457	0.000345328	0.508086	None	0

Project	Silva Valley Parkway OC		
Analysis Description			
Drawn By		*Scale* 1:200	*Company*
Date	10/27/2010, 7:43:08 AM	*File Name*	South OC Abutment.sli

rocscience

SLIDEINTERPRET 6.005



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	UCS (lb/ft2)	m	s	a	Water Surface	Ru
Foundation Concrete		155	Infinite strength							None	0
Structure Backfill		120	Mohr-Coulomb	0	34					None	0
Weathered Rock		170	Generalised Hoek-Brown			720000	0.584457	0.000345328	0.508086	None	0

rocscience	Project	Silva Valley Parkway OC
	Analysis Description	
	Drawn By	Scale 1:200.1 / Company
SLIDEINTERPRET 6.005	Date 10/27/2010, 7:43:08 AM	File Name North OC Abutment.slim

Project	Client	Page No. of
Silva OC	MTCo	1 1

Subject	File No.	By	Date
Type 1-Retaining Walls	556	PFF	4/16/12

Bearing Capacity for Retaining Wall

Type 1 retaining walls are to be located on the east side of abutment 3.

Design height: 14 to 18 ft.
Foundation Elev: 732.5 to 737.0 ft

For Type 1 wall, Case I condition, the maximum bearing stress is 2.6 to 3.1 Ksf at strength level (Caltrans standard Plan B3-1, 2010)

From AASHTO LRFD Bridge Design Specifications (2007), presumptive bearing resistance at the service limit state is in excess of 20 Ksf. (see table C10.6.2.6.1-1)

From previous analysis for abutment foundation (see this appendix) the nominal bearing resistance is approximately 50 tsf.

The factored resistance at the strength limit state is: 0.45 (50 tsf)
= 22.5 tsf

Bearing capacity is more than adequate for the proposed walls.

Settlement on competent rock is assumed to be less than 0.5 inches.

Note that review of rock discontinuities shows no detrimental orientations.

Checked by: RBS

APPENDIX E

Draft Report Comment and Response

OGDN Review Comment & Response Form

<table>
<tr><th>General Project Information</th><th colspan="2">Review Phase</th><th>Reviewer Information</th></tr>
<tr><td rowspan="3">Dist: 03
EA: 1E2901
EFIS Project No: 0300000258

Project Name:
Silva Valley Pkwy Interchange

Liaison Engineer:
Erick Fredrickson</td><td>☐ PSR/PDS (Review No. _)
☐ APS/PSR (Review No. _)
☐ APS/PR (Review No. _)
☐ Type Selection</td><td>☒ 65% PS&E Unchecked Details
☐ PS&E (Review No. _)
☐ Construction Support
☐ Other:</td><td rowspan="3">Reviewer: Thomas Song, PE
Functional Unit: 59-323 (Geotech North)
EFIS: 59-3657
Phone Number: (916) 227-1057
e-mail: Thomas_song@dot.ca.gov
Date of Review: 12/3/2010</td></tr>
<tr><th colspan="2">Structure Information</th></tr>
<tr><td>Structure Name
Silva Valley Pkwy OC
EB Off-Ramp UC
WB On-Ramp UC
WB Off-Ramp Br
WB Off-Ramp Retaining Wall
Carson Creek MSE Wall
Bucks Ravine Creek RCB</td><td>Bridge No.
25-0127
25-0128S
25-0129K
25-0130K</td></tr>
</table>

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
Patrick Fischer	Blackburn Consulting	530 887-1494	patf@blackburnconsulting.com	3/13/2012

Document Location (Page, Section, SSP)		OGDN Review Comment	Response	✓
1	General	This review includes the following documents: • The *Draft Foundation Reports, General Plans, Foundation Plans, Logs of Test Borings* for Silva Valley Pkwy OC (25-0127), Eastbound Off-Ramp UC (25-0128S), Westbound On-Ramp UC (25-0129K), and Westbound Off-Ramp Bridge (25-0130K). • The plans for Westbound Off-Ramp Retaining Wall, Carson Creek MSE Wall, and Bucks Ravine Creek RCB		

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved (for Reviewer's use)

2	Silva Valley Pkwy OC, Br. No. 25-0127 Foundation Plan, Sheet 3 of 26 and Log of Test Borings 1 of 4, Sheet 23 of 26	The plans indicate that the proposed construction will require approximately 20' or more of excavations in rock for abutments 1 and 3. Depending on the actual rock conditions, difficult excavation maybe encountered. Use of air tools or blasting may be required. This comment has been provided during Type Selection. If blasting is used, attention should be given to specifications that loose materials (blocks, etc) should be cleaned and cavities should be backfilled with structure concrete in footing excavations. This comment applies to some other structures too.	Rock excavation is addressed in the Geotechnical Design Report and additional commentary is added to the Foundation Report. Removal loose material and backfill of cavities is addressed in Section 11.1.	
3	Silva Valley Pkwy OC, Br. No. 25-0127, Draft Foundation Report, Appendix D, Calculations and Analyses, Bearing Capacity on Rock	Two values for Coefficient of Nms are shown. One value is identified as 0.024. Another value 0.05 is actually used in calculation.	Nms value of 0.05 is applicable and the calculation is corrected to be consistent.	
4	Silva Valley Pkwy OC, Br. No. 25-0127, Draft Foundation Report, Appendix D, Calculations and Analyses, Bearing Capacity on Rock	The conservatism and the related results are acceptable. It is reminded that BDS 4.4.8.1.2-1 may also be utilized with the Co being obtained from the lab results in Appendix C. This comment applies to some of other structures too.	Acknowledged. The Co value used is consistent with the lower Uniaxial (Unconfined) Compressive strength obtained on rock at this location.	
5	Silva Valley Eastbound Off-Ramp UC, Br. No. 25-0128S, Draft Foundation Report, Page 11, 12.1 Shallow Foundation	Please provide details for the usage of a modified bearing capacity factor, Nrq of 17.4.	Not applicable to the Silva Valley Pkwy OC	
6	Silva Valley Eastbound Off-Ramp UC, Br. No. 25-0128S, Draft Foundation Report, Page 11, 12.1.2 Lateral Resistance	There is no bent for this structure. For abutment footing, resistance factor should not apply since WSD is used.	Not applicable to the Silva Valley Pkwy OC	

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(for Reviewer's use)

7	Silva Valley Eastbound Off-Ramp UC, Br. No. 25-0128S, Draft Foundation Report, Appendix D, Design Calculations, Bearing Capacity	An internal friction angle of 38 degree might be too high for engineered backfill. This comment applies to other structures too.	Not applicable to the Silva Valley Pkwy OC	
8	Silva Valley Eastbound Off-Ramp UC, Br. No. 25-0128S, Draft Foundation Report, Appendix D, Design Calculations, Immediate Settlement of Spread Footing	Please provide details for the estimation of Es. This comment applies to other structures too.	Not applicable to the Silva Valley Pkwy OC	
9	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Page 10, Foundation Recommendations	The report indicates the subject structure is Silva Valley Eastbound Off-Ramp UC, which is another component structure of the project. Typo?	Not applicable to the Silva Valley Pkwy OC	
10	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Page 12, Table 5 - Foundation Design Recommendations for Spread Footings	Please provide details explaining the significant differences in recommendations for abutments 1 and 4.	Not applicable to the Silva Valley Pkwy OC	
11	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Page 12, 12.1 Shallow Foundation	Please provide details explaining the modified bearing capacity factor ($N\gamma q$ =19.2) used for bearing capacity of abutment 4. There is no discussion for abutment 1.	Not applicable to the Silva Valley Pkwy OC	
12	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Page 12, 12.1.2 Lateral Resistance	Is there any other lateral load(s) than seismic or other transient loads? This comment applies to some other structures too.	Not applicable to the Silva Valley Pkwy OC	

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13	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Appendix D, Design Calculation, Slope Stability Output	A friction angle of 38 degree is assigned to the fill materials, which is the same assigned for the Metavolcanic rock. The friction angle of 38 degree is too high for the fill materials.	Not applicable to the Silva Valley Pkwy OC	
14	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Appendix D, Design Calculation, Slope Stability Output	What groundwater condition is considered in the slope stability analyses? This comment applies to some other structures too.	Not applicable to the Silva Valley Pkwy OC	
15	Silva Valley Westbound Off -Ramp Bridge, Br. No. 25-0130K Draft Foundation Report Page 8, 10.0 Foundation Recommendations	The bottom elevations of the abutment footings are updated from what was provided during type selection, and both footings are split at the middle. Is the reason for splitting geotechnical design, ease of constructability, or other?	Not applicable to the Silva Valley Pkwy OC	
16	Silva Valley Westbound Off -Ramp Bridge, Br. No. 25-0130K Draft Foundation Report Page 9, 10.1.3 Lateral Resistance	The last sentence/statement in the first paragraph "… a passive earth pressure … neglect the upper 3 feet due to soil disturbance." may need to be further clarified. Since the passive earth pressure is against the vertical face of the footing, the 3-foot neglection maybe applicable to the bent footings due to their thickness of 4.5 feet. The thickness of the abutment footings is only 2.5 feet.	Not applicable to the Silva Valley Pkwy OC	
15	Silva Valley Westbound Off-Ramp Retaining Wall, General Plan No. 1, Sheet 1 of 6	The plan indicates there'd be more than 5 feet excavation to construct the wall footing, which may require temporary shoring. This comment applies to Carson Creek MSE Wall too.	Not applicable to the Silva Valley Pkwy OC	
16	Carson Creek MSE Wall General Plan, Sheet 1 of 8, TYPICAL SECTION	It is reminded that, for MSE wall founded on slopes, BDS 5.9.1 requires "A minimum horizontal beam of 4 feet or 0.1H (H is the wall height) wide, whichever is greater shall be provided in front of the wall."	Not applicable to the Silva Valley Pkwy OC	

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(for Reviewer's use)

17	Bucks Ravine Creek RCB, Double 6' X 7' RCB Details, Sheet 2 of 3, <u>AT CULVERT WINGWALLS</u>	The typical 2' of aggregate base (AB) immediately underneath the wing wall footings may need to be specified with a relative compaction requirement.	Not applicable to the Silva Valley Pkwy OC	

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(for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 12/01/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer to complete)
Dist: 03 EA: 1E2901 Project No: 0300000258 Project Name: Silva Valley Pkwy I/C OSFP Liaison: Eric Fredrickson Phone: 227-8916 e-mail: eric_fredrickson@dot.ca.gov	__ PSR/PDS (Review No.__ __ APS/PSR (Review No.__) __ APS/PR (Review No.__) __ Type Selection _X_ 65% PS&E Unchecked Details __ PS&E (Review No.__) __ Construction __ Other:________	Reviewer Name: __EDF_____ Functional Unit: ___OSFP____ Cost Center: _______ Phone Number: _______ e-mail: _______ Date of Review: 12-9-10 Structure Name*: _____var_______________ Br No*:_________ (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_______	MTCo.	_______	_______	_______

#	Doc. (See Note 1)	Page, Section, or SSP	ADDITIONAL FOUNDATION REPORT Review Comments	Consultant Responses	✓
1	FR #25-0127	Cover Pg	Revise "EA" to "03-1E2901"	Revision is made	
2		Pg 2	2nd para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2.2, 2nd para – Verify / update all bridge information w/ final plans.	Bridge number is included and all bridge information is updated with final plans provided by MTCo	
3		Pg 7	9.2 – Provide 'Mmax' used for ARS curve.	9.2: Mmax of 6.5 is provided.	
4		Pg 9	10, bullets – Verify / update all bridge information w/ final plans.	All bridge information is updated with final plans provided by MTCo	
5		Pg 10	Table 4 – Verify / update all bridge information w/ final plans. 10.1.2 – Provide commentary and recommendations about the plan for "future excavation" in front of Abutment 3 for Phase 2 construction. This difficult excavation will take place in front of the abutment (on spread footings), and under the bridge (low overhead). Should a significant portion of the future excavation take place during this stage of construction?	Table 4: All bridge information is updated with final plans provided by MTCo. 10.1.2: Commentary and recommendation is provided for future excavation in front of Abutment 3. Phase 1 excavation should consider access for future excavation.	

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✓= Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)

Dist-EA03-1E2901 Reviewer: __ EDF Str Name*: Silva Valley - various

Br No*.____ *=if applicable

6	**FR #25-0129K**	Cover Pg	Include PM. Revise "EA" to "03-1E290**1**"	Not applicable to the Silva Valley Pkwy OC	
7		Pg 1	2.1 – Revise the description from "100' south" to "xx' north". Be clear between 'old / existing' and 'new' Silva Valley Parkway.	Not applicable to the Silva Valley Pkwy OC	
8		Pg 2	1st para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2nd para – Delete 1st & 2nd sentence. 2.2, 3rd para – Verify side slopes at abutment 4 (4:1?).	Not applicable to the Silva Valley Pkwy OC	
9		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the Silva Valley Pkwy OC	
10		Pg 10	12 – Revise "EB Off-Ramp" with "WB On-Ramp"	Not applicable to the Silva Valley Pkwy OC	
11	**FR #25-0128S**	Cover Pg	Include PM. Revise "EA" to "03-1E290**1**"	Not applicable to the Silva Valley Pkwy OC	
12		Pg 1	2.1 – Be clear between 'old / existing' and 'new' Silva Valley Parkway.	Not applicable to the Silva Valley Pkwy OC	
13		Pg 2	1st para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2nd para – Delete 1st & 2nd sentence.	Not applicable to the Silva Valley Pkwy OC	
14		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the Silva Valley Pkwy OC	
15		Pg 10, 11	Table 3, 4, 5 – Revise / update footing 'L' dimension.	Not applicable to the Silva Valley Pkwy OC	
16	**FR #25-0130K**	Cover Pg	Revise "EA" to "03-1E290**1**"	Not applicable to the Silva Valley Pkwy OC	
17		Pg 2	2.2 – Revise bridge width dimension. 3 – Complete the description of the borings ("two…"borings?). Are there also "two" test pits?	Not applicable to the Silva Valley Pkwy OC	
18		Pg 6	9.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the Silva Valley Pkwy OC	
19		Pg 8, 9	10 – Revise / update abutment and bent footing dimensions.	Not applicable to the Silva Valley Pkwy OC	
20					
21	Ret Wall #3	General	Can this wall be eliminated with only slope excavation? R/W is available and existing side slopes are fairly steep with rocky material.	Not applicable to the Silva Valley Pkwy OC	
22					
23					
24					

Submittal Data (Reviewer to complete)

Dist-EA03-1E2901 Reviewer: __ EDF Str Name*: Silva Valley - various

Br No*.____ *=if applicable