FOUNDATION REPORT

Silva Valley Westbound
On-Ramp Undercrossing
El Dorado County, California
Bridge No. 25-0129K
03-ED-50
PM R1.65
EA 03-1E2901

Prepared for:

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May 14, 2012

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May 14, 2012

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Subject: **FINAL FOUNDATION REPORT**
Silva Valley Westbound On-Ramp UC
Bridge No. 25-0129K
03-ED-50; PM R1.65; EA 03-1E2901

Dear Ms. Passalacqua,

In accordance with our April 7, 2010 agreement, Blackburn Consulting (BCI) prepared this Final Foundation Report for the Silva Valley Westbound On-Ramp UC 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 26, 2010 and our Draft Foundation Report on November 8, 2010.

Please call if you have questions or require additional information.

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FINAL FOUNDATION REPORT
Silva Valley Westbound On-Ramp UC
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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Final Foundation Report for the new Silva Valley Westbound On-Ramp Undercrossing (UC) 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
- Prepared a Preliminary Foundation Report dated August 26, 2010 and a Draft Foundation Report dated November 8, 2010.
- Drilled and sampled one boring to a maximum depth of 29 feet below existing grade at Abutment 4 to supplement the nearby data from the US 50 Undercrossing
- 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
- Incorporated our responses to Caltrans review comments to the Draft Foundation Report (included in Appendix E).

This Foundation Report supersedes the referenced Preliminary and Draft Foundation Reports prepared by BCI.

2 PROJECT DESCRIPTION

2.1 Project Location and Site Description

The project is located in El Dorado County, California, along US 50 at Post Mile R1.65, approximately 20 to 30 feet north of the existing Clarksville Undercrossing (UC, at the existing Silva Valley Parkway). Figure 1 (Vicinity Map) in Appendix A shows the approximate project location.

Silva Valley Parkway is a two-lane (north/south) road that crosses under US 50, with no freeway access. The road is established in a “through-cut” section about 4 to 5 feet below the original ground surface. US 50 crosses over the road and is built upon 13 to 15 feet of embankment fill at the bridge abutments. The embankment end-slopes are unpaved at about 1½:1 (horizontal to vertical) and side-slopes are at 2:1.

The original US 50 bridges at Silva Valley Parkway (Clarksville UC, Bridge No. 25-0072 R/L) consist of two parallel bridges constructed in 1965. Each bridge is a 37-foot, 8-inch-wide by 110-foot-long, three-span structure. The substructure of each original bridge consists of open-style abutments supported on H-piles and two-column bents supported on spread footings. The original bridges were widened in 2010 with an infill at the median. For the infill project, the original foundation system was matched with H-Piles at the abutments and shallow spread footings at the bents.

The closest existing bridge structure is the Clarksville UC at Silva Valley Parkway. Vegetation consists primarily of moderately dense grasses and thistle, and a few small scattered trees near the abutment locations. Bents will be located on the road shoulder, which was cut down to grade. There are some underground utilities in this area.

2.2 Project Description

The project will consist of a new undercrossing structure, Silva Valley Westbound On-Ramp UC. The structure will be a three span, cast-in-place concrete voided slab bridge 115.5 feet long by 38.8 feet wide. The bridge will be located 20 to 30 feet north of the existing US 50 undercrossing. The new deck grade will be super elevated and will ascend from elev. 702.42 feet at Abutment 1 (Begin Bridge, “W3” Sta. 97+41.50) to elev. 707.09 feet at Abutment 4 (End Bridge, “W3” Sta. 98+57.00).

The substructure will consist of short-seat abutments and two, six-column bents all supported on spread footings. The abutment spread footings will be established within approach fill with uniform base of spread footing foundations planned at elevation 689.5 feet (about ½ to 1 foot below existing ground surface) at Abutment 1 and elevation 692.5 feet at Abutment 4 (about 3 feet above existing ground surface). The spread footings at Bent 2 and Bent 3 will be established within rock with uniform base of spread footing foundations planned at elevation 677.5 feet.

The new approach embankments will be about 12 feet high on the west (Abutment 1) and 21 feet high on the east (Abutment 4). The new embankment side/end-slopes will be constructed at 2:1 (horizontal:vertical distance), except at Abutment 4 where the north side-slope will be 4:1. The embankments will be constructed from material derived from cuts elsewhere within the project interval and/or other unknown sources.

Benchmark 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.

3 DOCUMENT REVIEW

BCI reviewed the following structure/site information for this project:

- Caltrans, Foundation Study, Clarksville Undercrossing III-ED-11-A, Br. #25-72 R&L, May 6, 1963.
- Caltrans, As-Built LOTB, Clarksville Undercrossing, Sheets 9 of 9, As-Built stamp undated, plans dated January 6, 1964.
- Caltrans, Memorandum, Foundation Report for Clarksville Undercrossing, August 3, 1965.
- Blackburn Consulting, Foundation Report for Clarksville UC (Widen), Bridge No. 25-0072L/R, EA 03-3A7111, El Dorado County, California, 2008.

4 SUBSURFACE EXPLORATION

To supplement the existing nearby boring data, further characterize the subsurface conditions and obtain samples for laboratory testing, BCI retained PC Exploration to drill and sample one exploratory boring (R-10-005) near the proposed Abutment 4 location. PC Exploration used a CME 75 truck-mounted rig to drill the boring on July 12, 2010 to a maximum depth of 30 feet below the ground surface (bgs). PC Exploration used hollow-stem auger to relatively competent bedrock, and then switched to HQ wireline diamond core equipment to complete the boring.

PC Exploration obtained relatively undisturbed samples using a Modified California Sampler. The sampler was driven into the ground with the force of a 140-pound hammer falling 30 inches using a hammer operated with an automated drop system. PC Exploration obtained rock cores by diamond-core barrel.

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 retained soil and rock samples recovered with the drive sampler in moisture-proof containers for laboratory testing and reference. Rock cores were retained in core boxes for reference. BCI also made groundwater observations in the borings during and at completion of drilling operations. At the completion of drilling, the boring was backfilled with cement-grout.

Appendix B contains the Log of Test Borings (LOTB) drawings for this project which provide more specific soil and rock descriptions and an explanation of descriptive terms used to log the soil and rock.

5 LABORATORY TESTING

BCI performed Unconfined Compressive Strength and Corrosivity (pH, Minimum Resistivity, Sulfates, and Chlorides) tests in the laboratory on some of the samples obtained from the exploratory boring.

We present the laboratory test results in Appendix C.

6 SITE GEOLOGY

6.1 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.

Most of El Dorado County is underlain by Mesozoic-age metavolcanic and metasedimentary rocks. The metamorphic rock structure is dominated by northwest trending foliation and northwest trending faults and fault zones that mark the boundaries of major rock types.

6.2 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) in Appendix A.

Rock structure at the UC location is expected to be similar to the surrounding area with predominant foliation having a strike of north, 35º to 45º west, and a steep dip of 70º-90º to the north.

We did not observe indications of slope instability on the natural slopes in the area. We did not observe groundwater seepage in the UC area.

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[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.

The West Bear Mountains Fault is located about 3,100 feet west of the site (near Latrobe Road) with a short splay mapped to the east approximately 1,200 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 UC 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 bridge 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.

7 SUBSURFACE CONDITIONS

7.1 Subsurface Soil and Rock Conditions

7.1.1 Caltrans (1963)

Subsurface exploration performed by the State Division of Highways (Caltrans) in April 1963 for the Clarksville UC consisted of four, 1-inch diameter soil tube borings. The foundation study and as-built Log of Test Borings (LOTB) drawing show subsurface materials encountered from original ground surface generally consist of 4 to 9 feet of stiff clay and slightly compact silty fine sand, underlain by sandstone, shale and schist. The foundation report states, "Approximately 17 feet of road embankment overlies the sand and clay at the right structure site." We include the as-built LOTB drawing in Appendix B.

7.1.2 BCI (2007)

BCI completed a total of five test borings in June/July 2007 for the Clarksville UC (Widen) project. In the existing UC abutment areas, subsurface materials generally consist of about 19 feet of roadway/embankment fill and native overburden materials comprised of medium dense and dense clayey gravel and silty sandy gravel (with local cobbles and boulders), and stiff to hard lean clay with varying amounts of sand and gravel. These materials are underlain by variably weathered and fractured metamorphic rock, consistent with published mapping. We include our LOTB drawings for the Clarksville UC (Widen) project in Appendix B.

[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

7.1.3 BCI (2010)

In Boring R-10-005 completed for this project element, BCI encountered metavolcanic rock at a depth of 3 feet. The rock is consistent with published mapping and previous site exploration. The upper 4 feet the rock between a depth of 3 to 7 feet is decomposed to intensely weathered and very intensely fractured (effectively soil-like described as very dense silty gravel). This portion of the rock was drillable using 6-inch diameter hollow-stem auger.

Below 7 feet to the maximum depth explored (30.0 feet) the rock is less weathered and required diamond coring for drill advancement. We generally describe rock within this interval as moderately to slightly weathered (locally intensely weathered and fresh), intensely to moderately fractured, and hard to very hard. The average core recovery was 99% and the Rock Quality Designation RQD[5] ranged from 0 to 92%.

The metavolcanic rock is overlain by 3 feet of residual soil comprised of hard silt with sand.

Refer to the LOTB and As-Built LOTB in Appendix B for more specific soil/rock descriptions, sampling methods, laboratory test results, and blow count data. We will include the required LOTB Sheet Checklist with the final report.

7.2 Groundwater

7.2.1 Caltrans (1963)

The Caltrans foundation study and as-built LOTB for the Clarksville UC indicate that static groundwater levels were measured at ground surface in one boring and a depth of about 2 feet in two of the borings completed in April 1963. The foundation study states, “This water is due to artesian flow from the underlying bedrock.”

The as-built LOTB identifies measured groundwater surface as follows:

Table 1 – Groundwater (Caltrans 1963 Exploration)

Boring	Ground Surface Elevation (ft)	Measured Ground Water Elevation (ft)
B2	686.6	684.5
B3	681.5	“Water flowing from B-3 at rate of ½ gal per minute.”
B4	676.5	676.5

Note: Elevations shown are referenced to datum used in 1963.

[5] RQD = Rock Quality Designation, expressed as the ratio of the total length of recovered rock core in pieces longer than 4-inches to the total length of core run.

The 1965 Foundation Report states, "Ground water was encountered at approximately 6' above the bottom of footing elevations. The footing excavations were dewatered by pumping for cleanup of the bottom of footings, forming and placing concrete."

7.2.2 BCI (2007)

During our June/July 2007 subsurface exploration for the Clarksville Undercrossing (Widen), BCI encountered groundwater at elev. 659.7, about 39 feet below ground surface in Boring R-07-B2. We did not encounter groundwater within the augered intervals in the other borings, and did not make groundwater measurements below the augered intervals due to the presence of residual drill fluid. None of the borings completed for the 2007 study exhibited artesian flow conditions.

7.2.3 BCI (2010)

We did not encounter free groundwater to elevation 683 feet within the augered portion of Boring R-10-005 drilled in July 2010. We did not make groundwater measurements below the augered interval due to the presence of residual drill fluid.

In general, we expect that shallow groundwater and seepage can occur near the soil/rock interface (depths of approximately 3 to 9 feet below existing, natural grade), particularly 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 or within excavations.

8 SCOUR EVALUATION

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

9 CORROSION EVALUATION

BCI evaluated one sample obtained during the 2010 site investigation for soil corrosivity. Table 2 presents the corrosivity test results.

Table 2 – Soil Corrosion Test Summary

Boring/Sample	Depth (ft)	Elevation (ft, msl)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
R-10-005 / S1B	1.0	689.0	3220	5.63	13.6	35.5

Note: Caltrans considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less. (Caltrans, "Corrosion Guidelines", version 1.0, September 2003)

Laboratory test results indicate a "non-corrosive" soils environment as defined by the September 2003 Caltrans "Corrosion Guidelines" publication. Laboratory tests results on two samples obtained during our 2007 site exploration for the Clarksville Undercrossing (Widen) project were also "non-corrosive." These laboratory test results are consistent with our previous study completed in 2008. Appendix C contains the laboratory test results for the 2010 study.

10 SEISMIC RECOMMENDATIONS

10.1 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)[6], 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 approximately 3,100 feet west of the bridge sites and a north-south trending splay associated with this fault crossing US 50 approximately 1,200 feet to the west. Jennings (1994)[7] shows the West Bear Mountains Fault as Pre-Quaternary in age. The Caltrans Deterministic PGA Map (September 2007) does not consider 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 bridge sites.

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

10.2 Ground Motion

BCI 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 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 750,000 years (late-Quaternary age) that are capable of producing a moment magnitude earthquake of 6.0 or greater. Caltrans procedures also require a minimum deterministic response spectrum that assumes a Maximum Moment Magnitude (MMax) of 6.5, vertical strike-slip event occurring at a distance of 7.5 miles.

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) located ±7 miles east of the site. Figure 3, Seismic Hazard Map, in Appendix A shows the approximate fault locations. Caltrans assigns the Bear Mountains Fault Zone (Rescue Fault section) the following parameters:

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

Table 3: Fault Data

Fault Parameters	Likely Fault
Fault Identification Number (FID)	83
Maximum Moment Magnitude (MMax)	6.5
Site-to-Fault (R_{RUP}) Distance (km/mi)	12.86 / 8.0
Style of Faulting	Normal
Fault Dip (degrees)	90
Dip Direction	Vertical

The probabilistic spectrum is obtained from the USGS (2008) National Hazard Map for 5% probability of exceedance in 50 years. Caltrans design spectrum is based 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 this site/project, we used a Site Class C with average Vs30 equal to 560 meters per second (approximately 1,800 feet per second) based on consideration of footings established in approach fill and the mapped ground conditions (underlain by metamorphic rock).

We recommend the design spectrum 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. BCI assigns the site a MMax of 6.5 with a Peak Ground Acceleration (PGA) of 0.21g. We present data points for site spectra in Table 4 and graphed site spectra on Figure 4.

Table 4 - Caltrans ARS Online Envelope* Spectrum Data

Period	SA	Period	SA	Period	SA	Period	SA
0	0.210	**0.085**	0.386	**0.35**	0.400	**1.4**	0.138
0.01	0.210	**0.09**	0.399	**0.36**	0.394	**1.5**	0.131
0.02	0.214	**0.095**	0.413	**0.38**	0.381	**1.6**	0.124
0.022	0.217	**0.1**	0.425	**0.4**	0.369	**1.7**	0.118
0.025	0.221	**0.11**	0.444	**0.42**	0.355	**1.8**	0.113
0.029	0.227	**0.12**	0.461	**0.44**	0.341	**1.9**	0.108
0.03	0.228	**0.13**	0.476	**0.45**	0.335	**2**	0.104
0.032	0.233	**0.133**	0.480	**0.46**	0.329	**2.2**	0.093
0.035	0.240	**0.14**	0.488	**0.48**	0.317	**2.4**	0.084
0.036	0.243	**0.15**	0.499	**0.5**	0.306	**2.5**	0.080
0.04	0.252	**0.16**	0.502	**0.55**	0.278	**2.6**	0.076
0.042	0.257	**0.17**	0.503	**0.6**	0.254	**2.8**	0.070
0.044	0.262	**0.18**	0.504	**0.65**	0.233	**3**	0.064
0.045	0.265	**0.19**	0.505	**0.667**	0.227	**3.2**	0.059
0.046	0.267	**0.2**	0.504	**0.7**	0.216	**3.4**	0.055
0.048	0.272	**0.22**	0.490	**0.75**	0.203	**3.5**	0.053
0.05	0.277	**0.24**	0.477	**0.8**	0.197	**3.6**	0.051
0.055	0.294	**0.25**	0.470	**0.85**	0.193	**3.8**	0.047
0.06	0.310	**0.26**	0.463	**0.9**	0.188	**4**	0.044
0.065	0.326	**0.28**	0.449	**0.95**	0.185	**4.2**	0.042
0.067	0.332	**0.29**	0.442	**1**	0.181	**4.4**	0.040
0.07	0.342	**0.3**	0.436	**1.1**	0.168	**4.6**	0.039
0.075	0.357	**0.32**	0.421	**1.2**	0.156	**4.8**	0.037
0.08	0.371	**0.34**	0.407	**1.3**	0.147	**5**	0.036

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

10.3 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 UC.

10.4 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.

10.5 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 failures on engineered fill slopes constructed at 1.5:1(horizontal: vertical) or flatter to be very low. We present further slope stability evaluation below in the Foundation Recommendations.

11 AS-BUILT DATA

A Caltrans April 5, 2000 Memorandum presents a summary of the existing Clarksville Road UC, Bridge No. 25-0072 L/R foundations. In general, the existing left and right bridges, constructed in 1965, consist of 3-span structures supported on a combination of spread footings and pile foundations. H-piles were used at the abutments and designed for a design load of 45 tons when driven to rock. Shallow spread footings were used at the bents and designed for an allowable bearing capacity of 5 tons per square foot (tsf). At the abutments, embankment fill was predrilled to elev. 680.0 and piles then driven using a Delmag D12 Diesel hammer. Rocks encountered during pre-drilling through the existing highway embankment slowed the drilling operations. At the left footing of Bent 3 (right bridge), excavation was difficult and blasting was required to achieve the planned footing level.

BCI (2008) provided foundation recommendations for the bridge widening (to the median) at the Clarksville UC. The existing foundation system was matched with H-Piles at the abutments and shallow spread footings at the bents. H-piles were designed for a nominal resistance of 170 kips when driven to rock. Shallow spread footings on rock were designed using a Net Permissible Contact Stress of 23.0 to 31.5 kips per square foot.

12 FOUNDATION RECOMMENDATIONS

The new abutments for the Silva Valley Westbound On-Ramp UC will be founded on shallow spread footings established within engineered fill. The new bents will be established within moderately to slightly weathered, hard rock at least 6.5 feet below lowest existing grade.
The base of the spread footing at Abutment 1 will be about ½ to 1 foot below existing ground surface and at Abutment 4 about 2.5 feet above existing ground surface. To provide uniform support and minimize post construction settlement of footings founded in fill, we recommend that the abutment footings be established within a prism of engineered fill

At Abutment 1 overexcavate all existing fill and native overburden materials to elev. 685.5 feet; at Abutment 4 to elev. 688.5 feet. Replace the overexcavated materials to footing grade with engineered "Structure Backfill" (per Section 19 of Caltrans "Standard Specification") compacted to a minimum of 95% relative compaction (per CTM 216). Extend the limits of the engineered fill prism to at least 5 feet horizontally beyond the footing footprint.

We considered Cast in Drilled Hole (CIDH) pile foundations or large diameter drilled-shafts; however, casing would be required in the fill section at Abutment 4 and difficult drilling is expected due to both the hardness of the underlying rock and the frequency of fractures. Driven concrete piles are not an appropriate foundation alternative. Such piles 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. H-piles, similar to the nearby widened structure, would also experience very hard driving in rock, be essentially point bearing, and have very limited lateral capacity. Therefore, we do not recommend H-piles for new bridge support.

MTCo provided the following foundation design information in Tables 5, 6 and 7.

Table 5 - Foundation Data

Support No.	Design Method	Finish Grade Elev. (ft)	BOF Elevation (ft)	Footing Size (ft)		Permissible Settlement under Service Load (in) *
				B	L	
Abut 1	WSD	694.4	689.5	8.0	40.9	1.0
Bent 2	LRFD	684.0	677.5	8.5	42.0	1.0
Bent 3	LRFD	684.0	677.5	8.5	42.0	1.0
Abut 4	WSD	697.4	692.5	8.0	40.9	1.0

*Based on CALTRANS' current practice, the total permissible settlement for a shallow footing is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Table 6 - LRFD Service Limit State I

Support No.	Total Load				Permanent Load *		
	Vertical Load (kip)	Effective Dimensions (ft)		Horizontal Load in Longitudinal Direction (kip)	Vertical Load (kip)	Effective Dimensions (ft)	
		B'	L'			B'	L'
Abut 1	760	7.2	40.9	160	610	7.3	40.9
Bent 2	1050	7.2	39.4	N/A	820	8.3	42.0
Bent 3	1060	7.1	37.4	N/A	820	8.4	41.9
Abut 4	820	7.4	40.9	190	670	7.0	40.9

* See table 3.4.1-2 in the AASHTO LRFD Bridge Design Specifications for components of permanent load. Total and Permanent Loads are NET for Bents and GROSS for Abutments.

Table 7 - LRFD Strength and Extreme Event Limit States

Support No.	Strength Limit State (Controlling Group)			Extreme Event Limit State (Controlling Group)		
	Vertical Load (kip)	Effective Dimensions (ft)		Vertical Load (kip)	Effective Dimensions (ft)	
		B'	L'		B'	L'
Bent 2	1440	6.6	38.0	820	8.3	42.0
Bent 3	1460	5.9	35.7	830	8.4	41.9

12.1 Shallow Foundations

1.1.1 Spread Footing Data Table

Based on footing foundation design data provided by MTCo and our geotechnical analysis, we provide foundation design recommendations in Table 8. A discussion of our analyses follows.

Table 8 – 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	8.0	40.9	689.5	4.9	6.40	4.58	N/A	N/A	N/A
Bent 2	8.5	42.0	677.5	6.5	N/A	N/A	25.00	16.83	37.41
Bent 3	8.5	42.0	677.5	6.5	N/A	N/A	26.60	16.04	35.64
Abut 4	8.0	40.9	692.5	4.9	7.70	5.40	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, where applicable.
2) See Memo to Designers (MTD) 4-1 for definitions and applications of the recommended design parameters.

At the bents, we conservatively modeled the weathered/fractured rock (RQD < 25%) as a very dense soil with a friction angle of 38° and no cohesion. At the abutments, BCI used a friction angle of 34° with no cohesion for engineered fill and determined a modified bearing capacity factor ($N_{\gamma q}$) for the abutment footings established adjacent to sloping ground based after Meyerhof (1957) which assumes cohesionless soils. We modeled ground water at elev. 684.0 ft. We include our spread footing design calculations, including determination of $N_{\gamma q}$ at the abutments, in Appendix D.

12.1.1 Slope Stability

The base of footing at Abutment 4 will be founded within new embankment fill about 3 ft above original ground surface and with a maximum proposed slope gradient of 2(H):1(V) in front of the abutment.

We evaluated Abutment 4 established in new embankment for global stability with respect to static loading and pseudostatic (seismic) loading conditions. For pseudostatic conditions we used a horizontal seismic acceleration coefficient of 0.1.

BCI used SLIDE 6.0 limit equilibrium slope stability software by Rocscience, Inc. to analyze slope stability. We analyzed the cross-section using the Spencer method of slices, which satisfies both force and moment equilibrium, and circular shaped failure surfaces. Considering local material types, we anticipate that coarse granular materials with a silt/clay matrix will be used for new embankment. For our slope stability analysis, we modeled the new embankment fill with an angle of internal friction equal to 34° and a nominal cohesion value of 50 psf. We modeled the underlying decomposed and very intensely fractured rock with a friction angle of 40°; moderately weathered rock with a friction angle of 43°. We modeled ground water at elev. 684.0 ft.

The computed slope stability factor of safety for static loading is 1.5, and for pseudostatic loading is 1.3. We expect conditions at Abutment 1 to be the same or better.

We include the graphical outputs from our stability trials that show soil/rock parameters and foundation loading conditions used in our analysis in Appendix D.

12.1.2 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 engineered fill. Use a friction angle (ϕ_f) of 34° and 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 friction angle (ϕ_f) of 34° and a resistance factor (φ_{ep}) of 0.5 for LRFD.
- Passive and friction resistance may be combined.

12.1.3 Settlement

We calculated the settlement of spread footing foundations based on elastic settlement theory using Schmertmann's Modified Method. We conservatively modeled the underlying rock at all supports as a very dense soil. For spread footings established as above, we estimate that settlement will be nominal (about ½-inch or less) and will occur substantially during construction. We expect differential settlement to be less than one-half of the total realized settlement. We include our abutment settlement calculations in Appendix D.

12.2 Approach/Abutment Backfill Earthwork

12.2.1 Fill Material

The source of borrow material for construction of approach fills has not been identified. Proposed borrow must be tested and approved for use by the project engineer prior to transporting to the site.

12.2.2 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).

12.2.3 Geometry and Stability

The maximum fill height at the bridge abutments will range from about 12 to 21 feet. Approach side-slopes will have a gradient of 2:1 or flatter and the end-slopes will have a gradient of 2:1. The proposed geometries are common slope gradients considered stable for typical approach fill construction.

In our opinion, the proposed new side/end-slopes will be stable provided the new slopes are constructed in accordance with current Caltrans Standard Specifications. The generally hard/dense nature of the underlying native soil and rock will provide a stable base on which to construct the fills.

12.2.4 Site Preparation

In the area of the proposed 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. The project geotechnical engineer must approve the prepared ground surface prior to placement of approach fill.

At Abutment 1 subexcavate all existing fill and native overburden materials to expose intact weathered rock and replace to footing grade with engineered "Structure Backfill" (per Section 19 of Caltrans "Standard Specification") compacted to a minimum of 95% relative compaction (per CTM 216). Extend the limits of the engineered fill prism to at least 3 feet below the base of footing and horizontally 5 feet beyond the footing footprint.

12.2.5 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 "Standard Specifications." A waiting period is not necessary.

12.2.6 Lateral Earth Pressures

Use the following EFWs to design the abutments walls and wing walls at Abutments 1 and 4:

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 drainage behind walls 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 0.11 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 compacted fill materials. The passive pressures are applicable for concrete placed directly compacted fill.

13 CONSTRUCTION CONSIDERATIONS

13.1 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 undercrossing. 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.

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 formed by the blocks with structural concrete.

13.2 Embankments

Import borrow sources are not yet identified and, therefore, imported embankment materials cannot be evaluated. We expect slopes constructed of on-site materials or imported borrow to meet the specifications for embankment fill, and sloped at a gradient of 2(h):1(v) or flatter, to be grossly stable. Material used for backfill at abutments must meet the requirements for Structure Backfill.

13.3 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.

A BCI geologist or engineer must review foundation excavations to confirm suitable bearing material and/or identify loose/soft or unsuitable materials to be overexcavated.

13.4 Dewatering

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

13.5 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.

Based on the preliminary test results (BCI, 2008), and the unknown origin of fill placed during road construction in the 1960’s (and previous), 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 construction activities 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.

13.6 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.

14 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 monitor footing excavations, 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.

15 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/groundwater conditions we observed in our borings are representative of the subsurface conditions on the site. Actual conditions between borings could be different.

Our scope did not include an evaluation of potential flooding or hazardous materials on site.

Use this foundation report only for the design and construction of the Silva Valley Westbound On-Ramp UC.

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

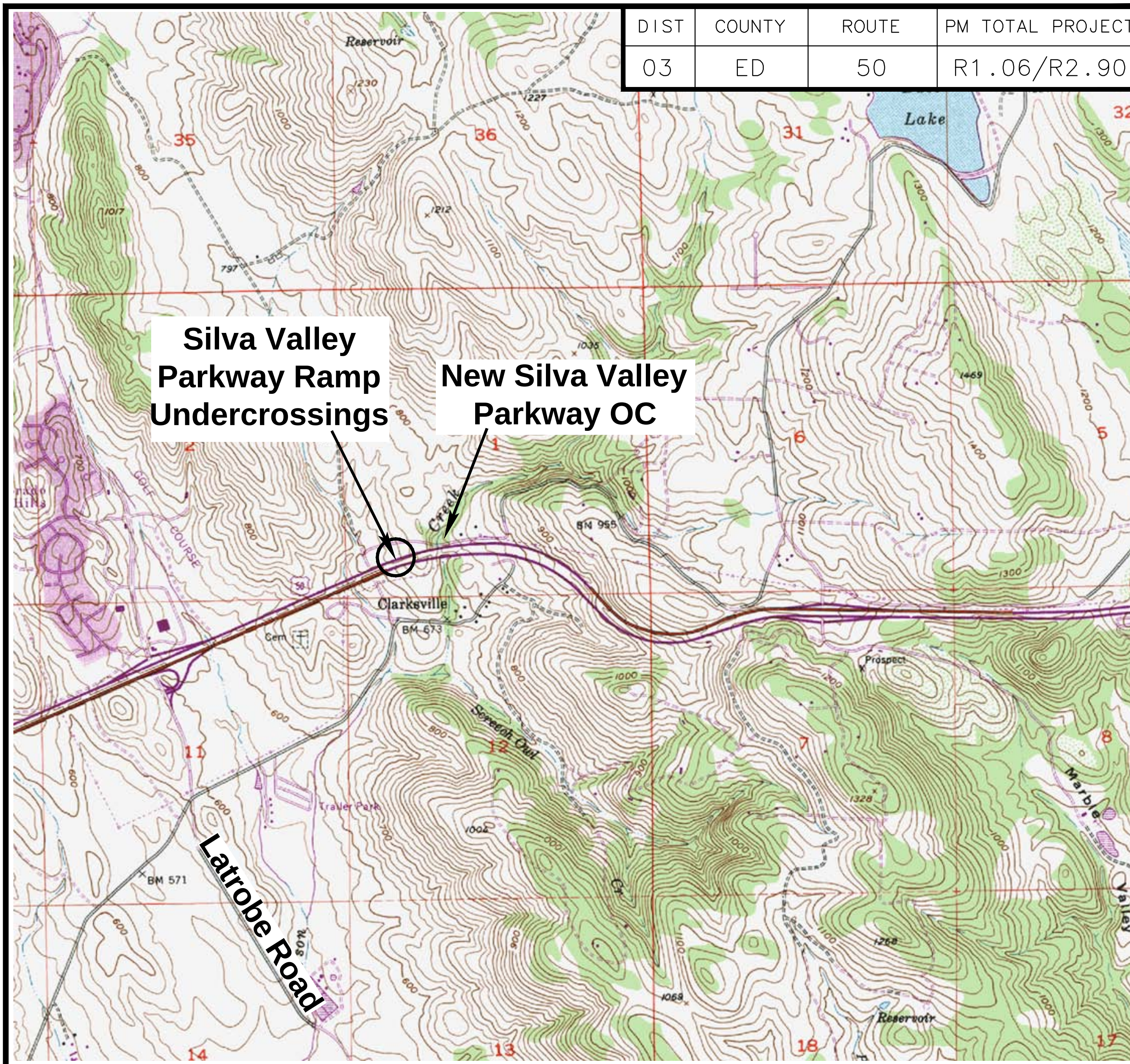
Figure 1: Vicinity Map

Figure 2: Regional Geologic Map

Figure 3: Seismic Hazard Map

Figure 4: ARS Curve

DIST	COUNTY	ROUTE	PM TOTAL PROJECT
03	ED	50	R1.06/R2.90



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

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www.blackburnconsulting.com

VICINITY MAP

Silva Valley WB On-Ramp UC
EA 03-1E2901
El Dorado County, California

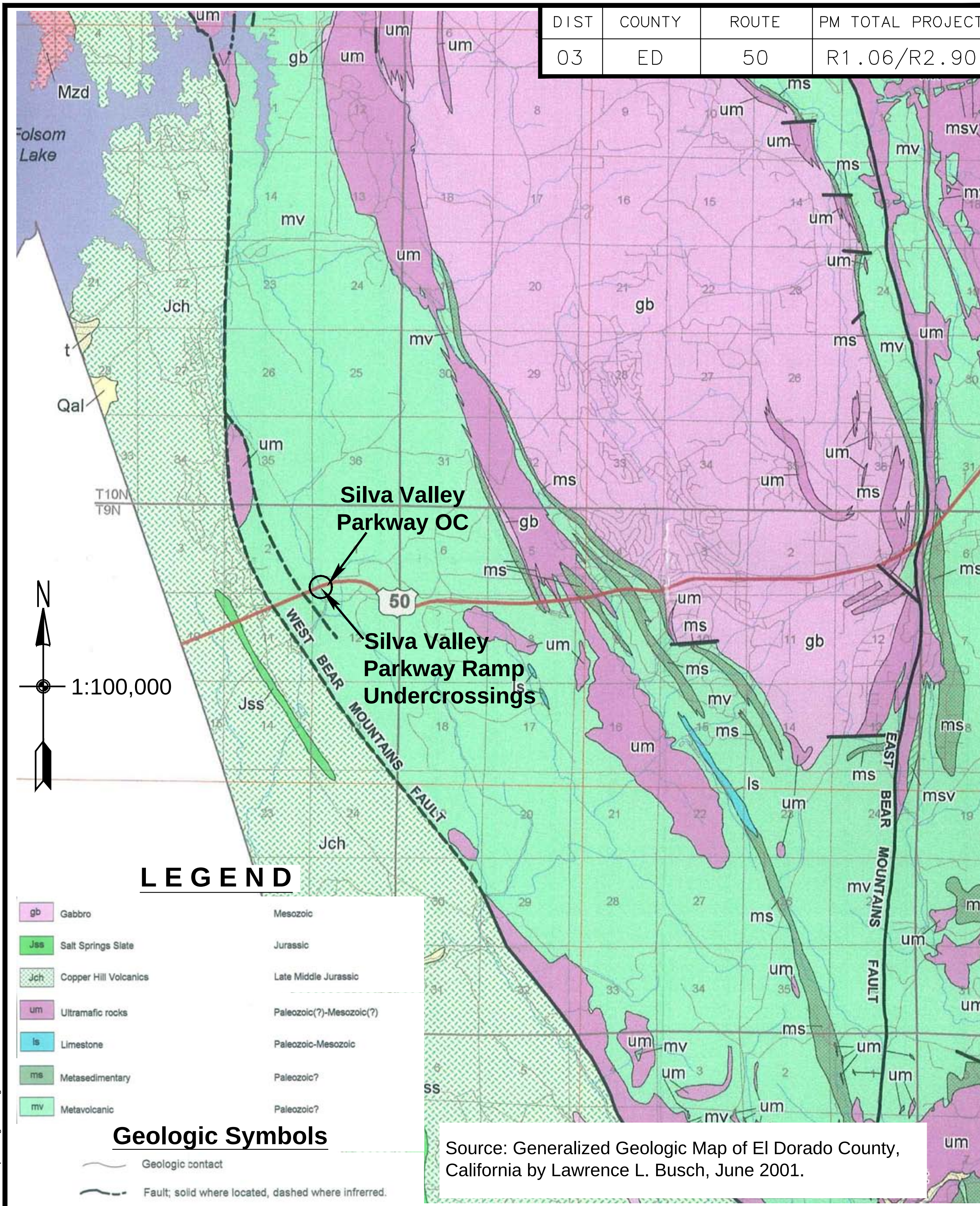
File No. 556.2

May 2012

Figure 1

5/11/2012 556.2 Silva Valley WB On-Ramp UC Figure 1.dwg

DIST	COUNTY	ROUTE	PM TOTAL PROJECT
03	ED	50	R1.06/R2.90



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GEOLOGIC MAP

Silva Valley WB On-Ramp UC
EA 03-1E2901
El Dorado County, California

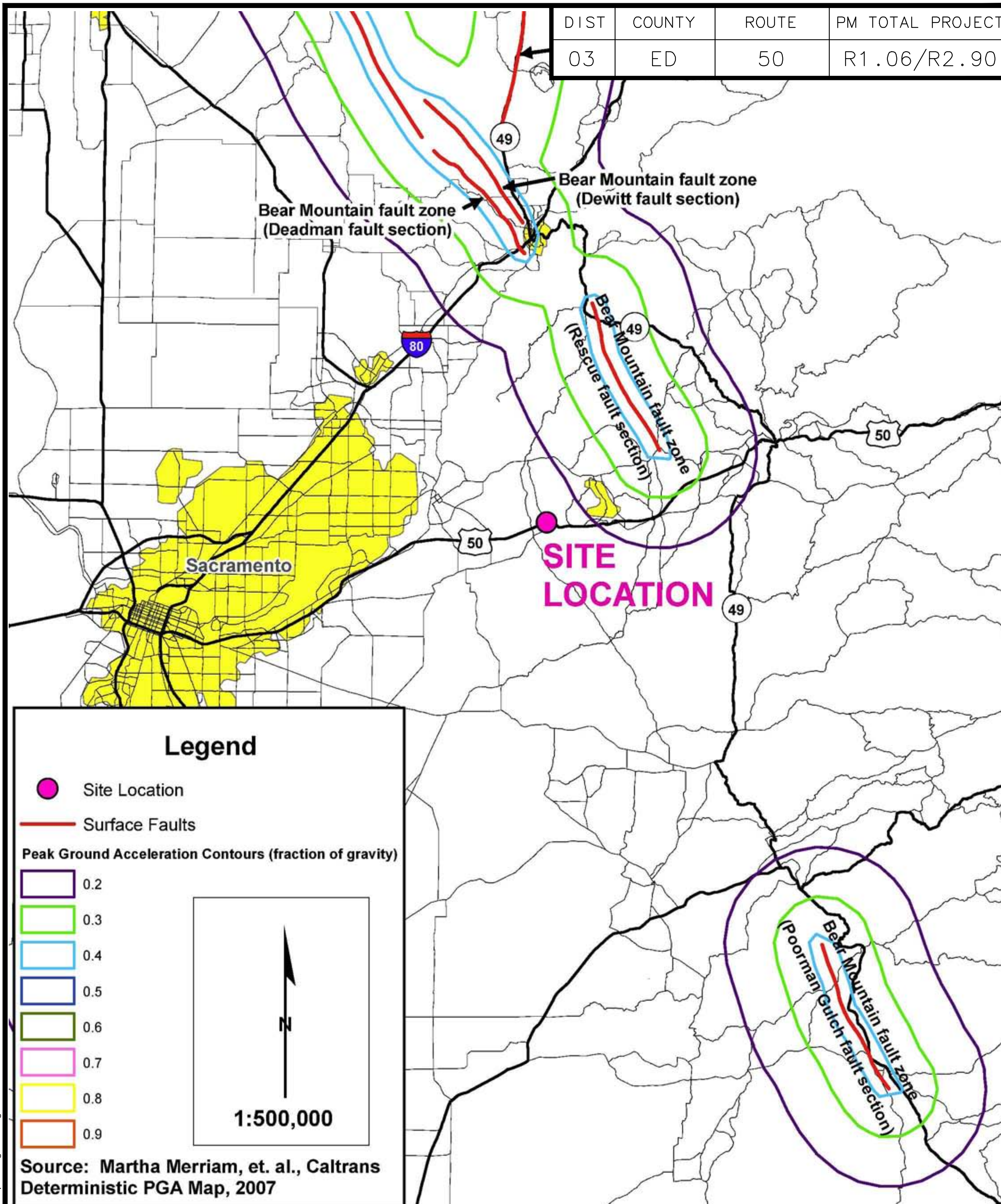
File No. 556.2

May 2012

Figure 2

5/11/2012 556.2 Silva Valley WB On-Ramp UC Figure 2.dwg

DIST	COUNTY	ROUTE	PM TOTAL PROJECT
03	ED	50	R1.06/R2.90



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SEISMIC HAZARD MAP

Silva Valley WB On-Ramp UC
EA 03-1E2901
El Dorado County, California

File No. 556.2

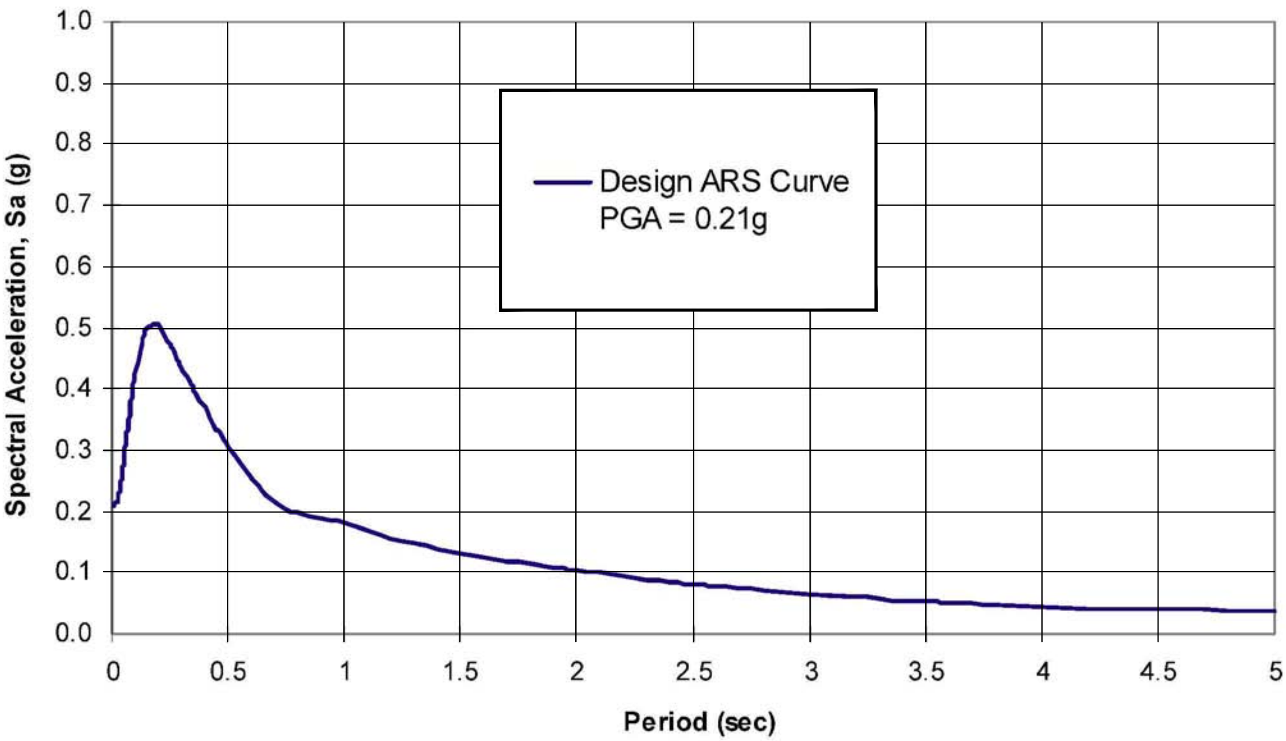
May 2012

Figure 3

5/11/2012 556.2 Silva Valley WB On-Ramp UC Figure 3.dwg

DIST	COUNTY	ROUTE	PM TOTAL PROJECT
03	ED	50	R1.06/R2.90

Design ARS Curve
(5% Damping)



Reference: Geotechnical Services Design Manual (Version 1.0, August 2009) and Caltrans Seismic Design Criteria, Appendix B, Revised 9/11/2009.

11/5/2010 556.2 Silva Valley WB On-Ramp UC Figure 4.dwg

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PRELIMINARY ARS CURVE
Silva Valley WB On-Ramp UC
EA 03-1E2901
El Dorado County, California

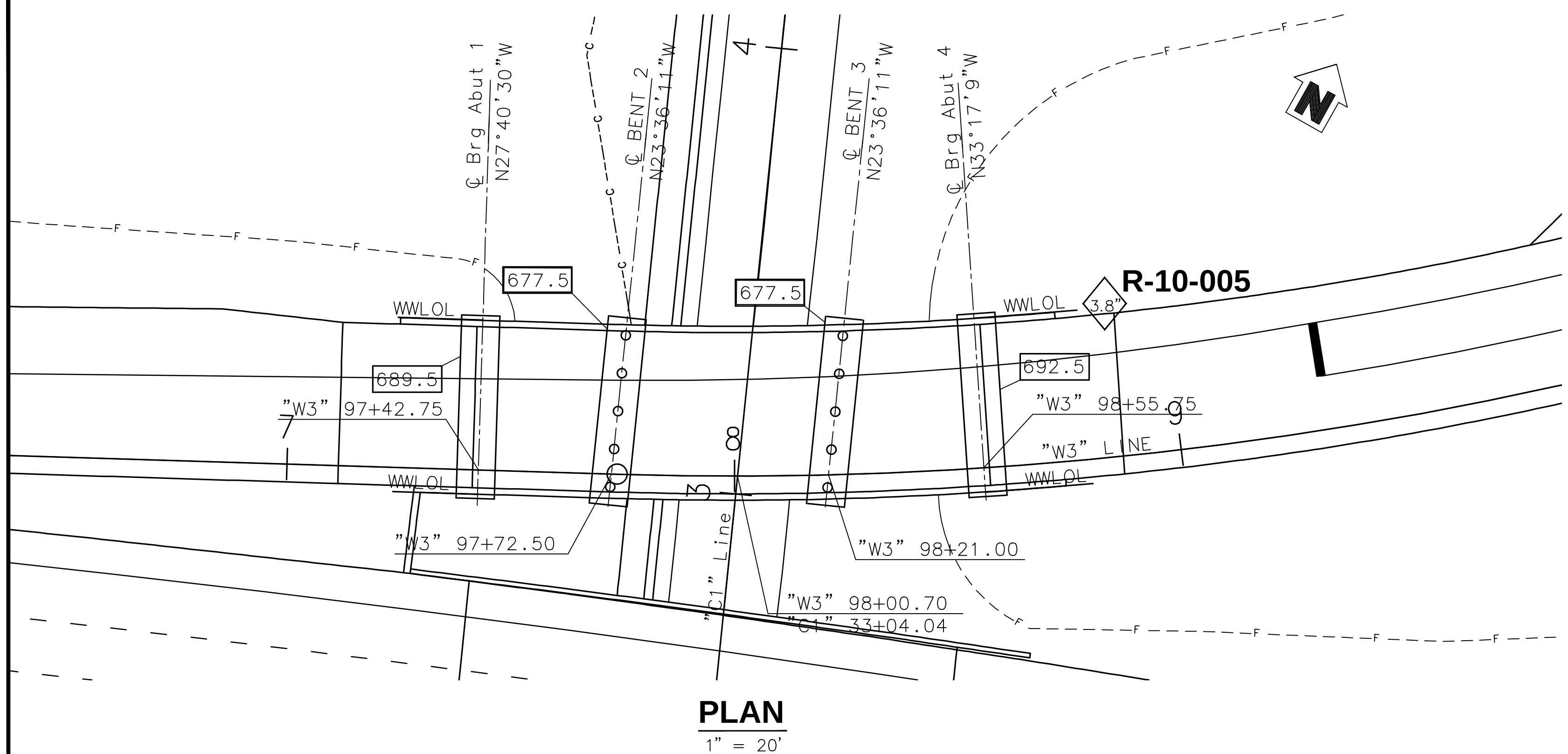
File No. 556.2

May 2012

Figure 4

APPENDIX B

Log of Test Borings (7 sheets)



PLAN

1" = 20'

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. 3, and 4, "Soil Legend" and 5, "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 03/2012.
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".

BENCHMARK

Orthometric Heights (elevations) shown are NGVD 29. Based on HPGN D CA 03 DL having an elevation of 693.55 and USGS BMT 127 (PID JS0692) having and elevation of 673.08.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.07/R2.40	296	349

CERTIFIED ENGINEERING GEOLOGIST 05/16/12 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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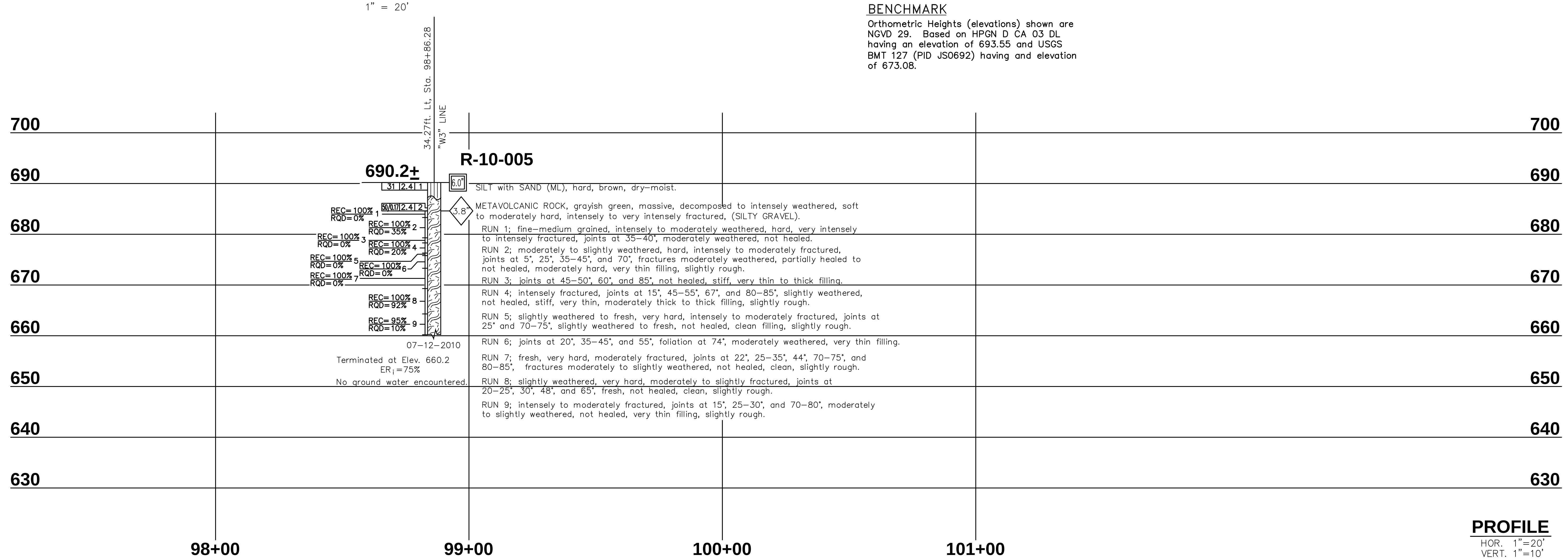
COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
2850 FAIRLANE COURT
PLACERVILLE, CA 95667

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



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. 25-0129K
POST MILE R1.65

SILVA VALLEY WB ON-RAMP UC
LOG OF TEST BORINGS 1 OF 7

GS GEOLOGIST LOG OF TEST BORINGS 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 03/12/12 05/16/12 SHEET 15 OF 21

FILE => $REQUEST

5/16/2012 556.2 Silva Valley WB On-Ramp UC LOTB.dwg

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

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	1.07/R2.40	297	349

CERTIFIED ENGINEERING GEOLOGIST 05/16/12 DATE

PLANS APPROVAL DATE

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

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COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
2850 FAIRLANE COURT
PLACERVILLE, CA 95667

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

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

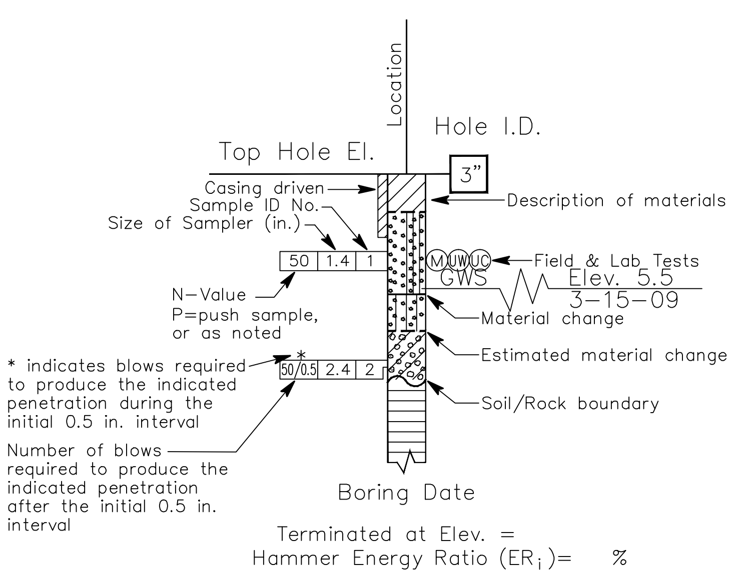
BOREHOLE IDENTIFICATION

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

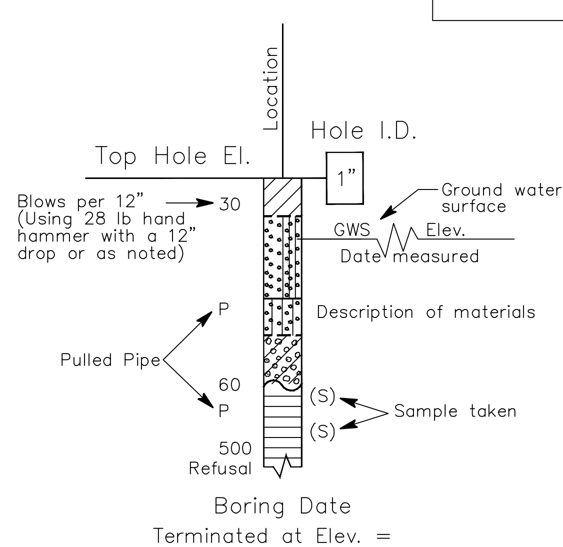
NOTE: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

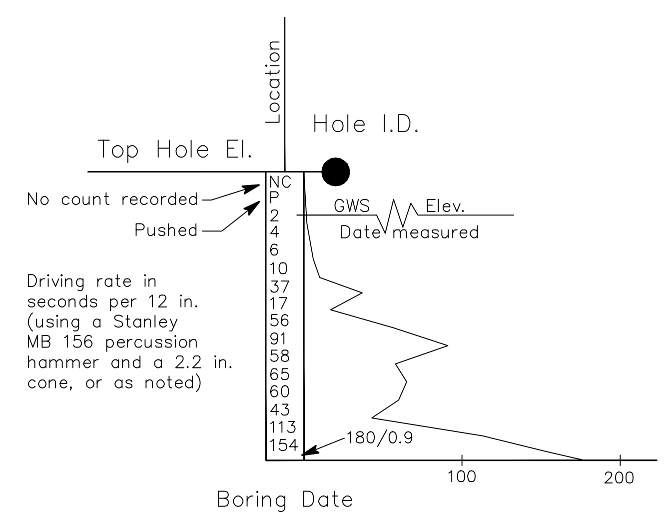
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be 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.



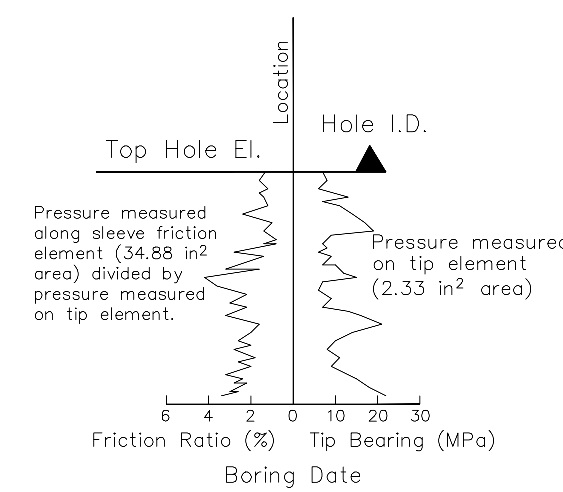
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

SOIL LEGEND

SILVA VALLEY WB ON-RAMP UC

LOG OF TEST BORINGS 2 OF 7

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. 25-0129K
POST MILE R1.65

GS LOTB SOIL 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

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 03/02/12 05/16/12 SHEET 16 OF 21

FILE => $REQUEST

5/16/2012 556.2 Silva Valley WB On-Ramp UC LOTB.dwg

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

CERTIFIED ENGINEERING GEOLOGIST 05/16/12 DATE

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

PLANS APPROVAL DATE

COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
2850 FAIRLANE COURT
PLACERVILLE, CA 95667

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

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 WB ON-RAMP UC

LOG OF TEST BORINGS 3 OF 7

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	1.07/R2.40	299	349

CERTIFIED ENGINEERING GEOLOGIST 05/16/12 DATE

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

PLANS APPROVAL DATE

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

COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
2850 FAIRLANE COURT
PLACERVILLE, CA 95667

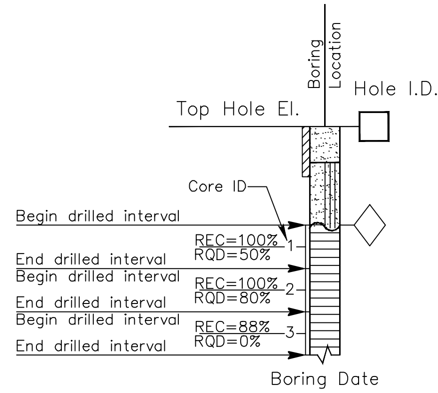
BLACKBURN CONSULTING
11521 BLOCKER DRIVE, SUITE 110 FILE No. 556.2
AUBURN, CA 95603

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: Chemical weathering–Discoloration and/or oxidation: Body of rock	Chemical weathering–Discoloration and/or oxidation: Fracture Surfaces	Mechanical Weathering–Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and solutioning: Texture	Texture and solutioning: Solutioning	General Characteristics
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 discolorization 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 hairline 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 WB ON-RAMP UC

LOG OF TEST BORINGS 4 OF 7

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. 25-0129K
POST MILE R1.65

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

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES: 03/12/12 05/16/12 — SHEET 18 OF 21

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

5/16/2012 556.2 Silva Valley WB On-Ramp UC LOTB.dwg

FILE => $REQUEST

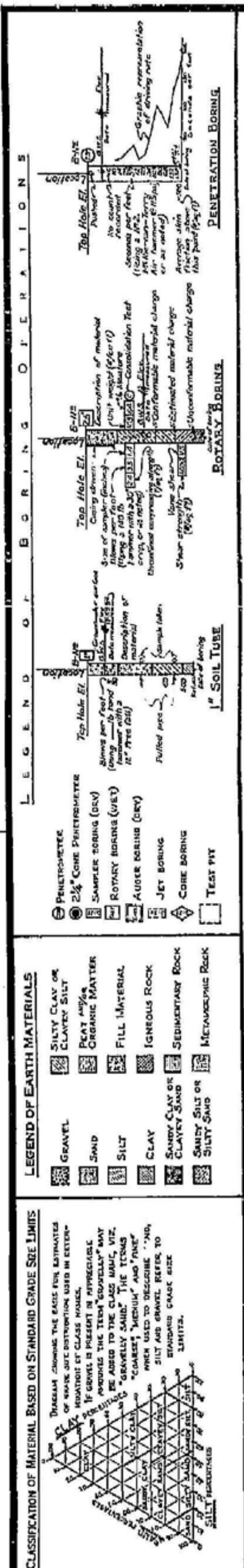
USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

III-ED-50

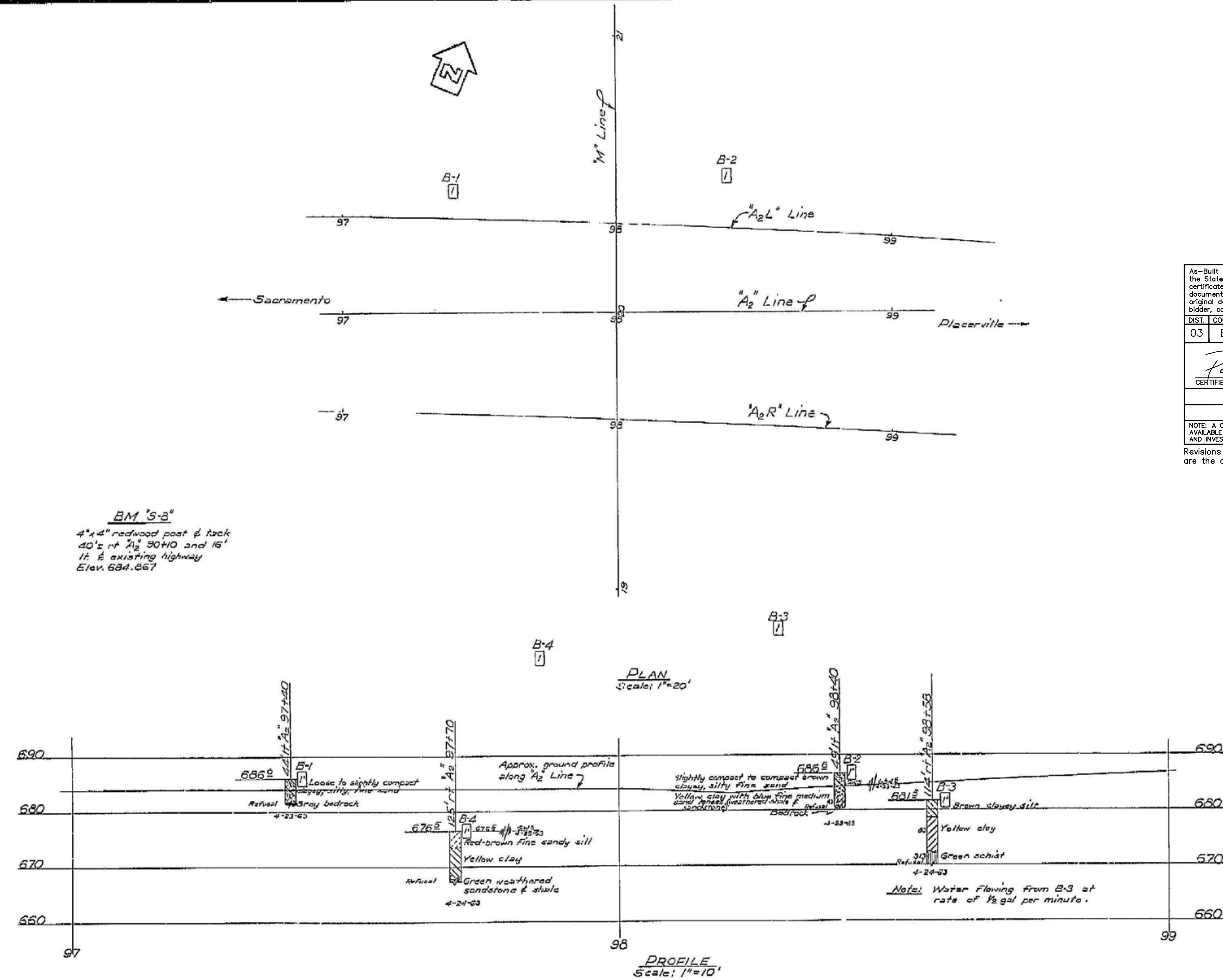
FED. ROAD DIV. No.	STATE	F.A. PROJECT No.	SHEET No.	TOTAL SHEETS
7	CALIF.			

DIST.	COUNTY	ROUTE	SECTION	SHEET No.	TOTAL SHEETS
III	ED	4	A	171	171

DATE APPROVED January 6, 1964



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	POST MILES—TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	ED	50	1.07/R2.40	302	349

CERTIFIED ENGINEERING GEOLOGIST 05/16/12 DATE

SILVA VALLEY WB ON-RAMP UC

LOG OF TEST BORINGS 7 OF 7

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA.

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

Revisions shown on this Log of Test Borings are the addition of the following table and notes.

BRIDGE NO.	SHEET	OF
25—0129K	21	21

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

INFORMATION ON ACTUAL FOUNDATION CONDITIONS ENCOUNTERED IS ON FILE IN BRIDGE GEOLOGY SECTION

THIS SET OF PLANS HAS BEEN CORRECTED TO CORRESPOND TO THE "AS BUILT" PLANS DATED ______, AS SUBMITTED BY RESIDENT ENGINEER ______
TRACINGS CORRECTED BY: ____ DATE: ____

FIELD STUDY	By Allison	4-63
DRAWN	By Darr	4-63
CHECKED	By Allison	4-63

BRIDGE DEPARTMENT
ENGINEERING GEOLOGY SECTION

171

AS BUILT PLANS
Contract No. 03-074024
Date Completed ______
Document No. 30001384

SHEET	OF
9	9

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

CLARKSVILLE UNDERCROSSING

LOG OF TEST BORINGS

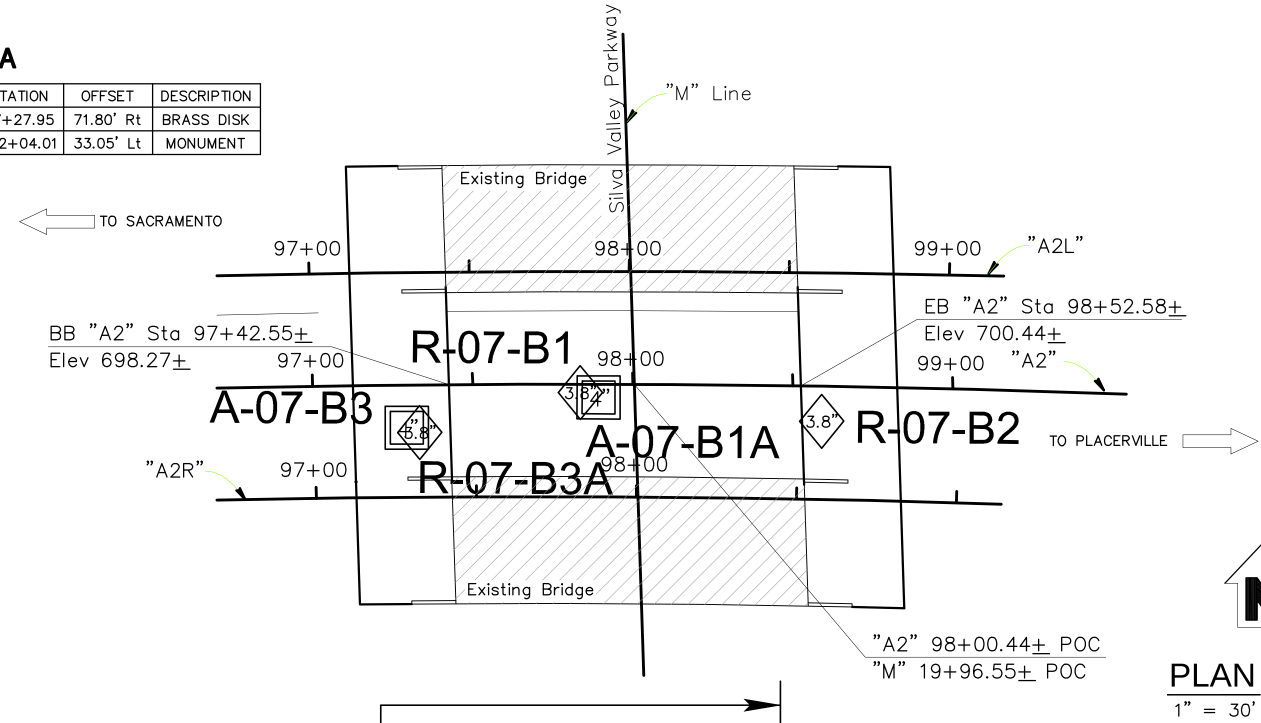
SCALE As shown | BRIDGE 25-72 R/L | FILE | DRAWING 25.72-9

Disregard prints bearing earlier numbers → PREL. DRAWING No. PR-

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO

BENCH MARK DATA

NO.	NORTHING	EASTING	ELEVATION	LINE	STATION	OFFSET	DESCRIPTION
25124	2,002,545.43	6,830,886.32	695.47	"A2"	97+27.95	71.80' Rt	BRASS DISK
517	2,002,837.42	6,831,282.78	711.28	"A2L"	102+04.01	33.05' Lt	MONUMENT



PLAN
1" = 30'

NOTES:
1. Field classification of soil and rock was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2007). See Log of Test Borings 4 of 5 "Soil Legend" and Log of Test Borings 5 of 5 "Rock Legend".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a 140 lb. safety hammer operated with cat-head, rope and pully with a 30-inch drop. 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 but with brass liners.
4. 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.
5. 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 Caltrans Soil & Rock Logging, Classification, and Presentation Manual.
6. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 2007).
7. Where less than 0.5 feet of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
8. Consistency of soils shown in ( ) where estimated.
8. Ground water surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
9. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
10. Electronic media for preliminary plan view provided by Quincy Engineering, Inc., dated August 21, 2007.
11. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".
12. This LOTB was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	0.0-2.9	447	451

08/11/08
CERTIFIED ENGINEERING GEOLOGIST DATE

PROFESSIONAL GEOLOGIST William E. Nichols No. 2229 Exp. 1-31-10 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

PLANS APPROVAL DATE

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

BLACKBURN CONSULTING
2437 FRONT STREET
WEST SACRAMENTO, CA 95691
File No. 1072.2

QUINCY ENGINEERING
3247 RAMOS CIRCLE
SACRAMENTO, CA 95827-2501

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	POST MILES-TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	ED	50	1.07/R2.40	300	349

05/16/12
CERTIFIED ENGINEERING GEOLOGIST DATE

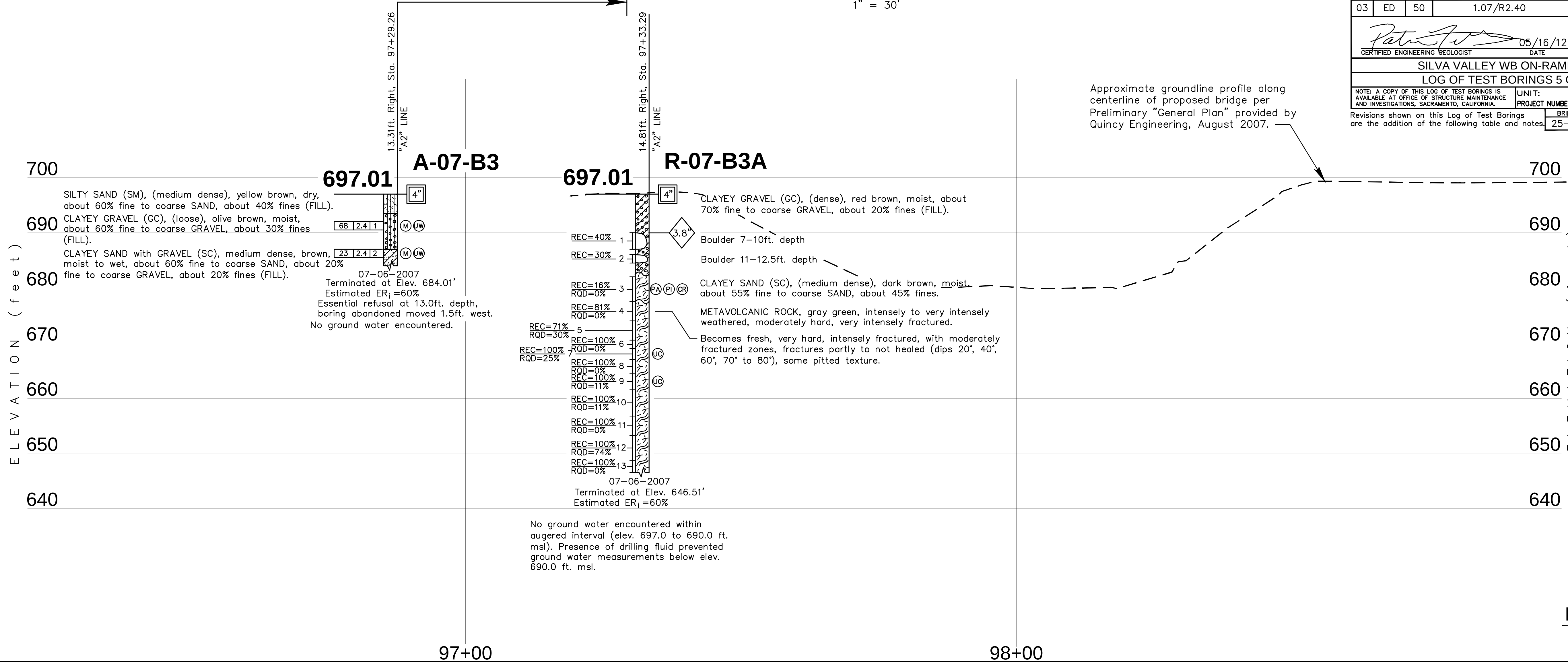
SILVA VALLEY WB ON-RAMP UC
LOG OF TEST BORINGS 5 OF 7

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA.
UNIT: 03-1E2901
PROJECT NUMBER & PHASE: 0300000258

Revisions shown on this Log of Test Borings are the addition of the following table and notes.

BRIDGE NO.	SHEET	OF
25-0129K	19	21

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



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

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DESIGN OVERSIGHT	BRIDGE NO. 25-0072	CLARKSVILLE UNDERCROSSING (WIDEN)
FUNCTIONAL SUPERVISOR	DRAWN BY: M. D. Robertson	FIELD INVESTIGATION BY:		NAME: Eric Fredrickson	POST MILE 1.66	LOG OF TEST BORINGS 1 of 5
NAME:	CHECKED BY: W. E. Nichols	Rob Pickard, June 2007				

04/22/08 1072.2 Clarksville Road UC.dwg

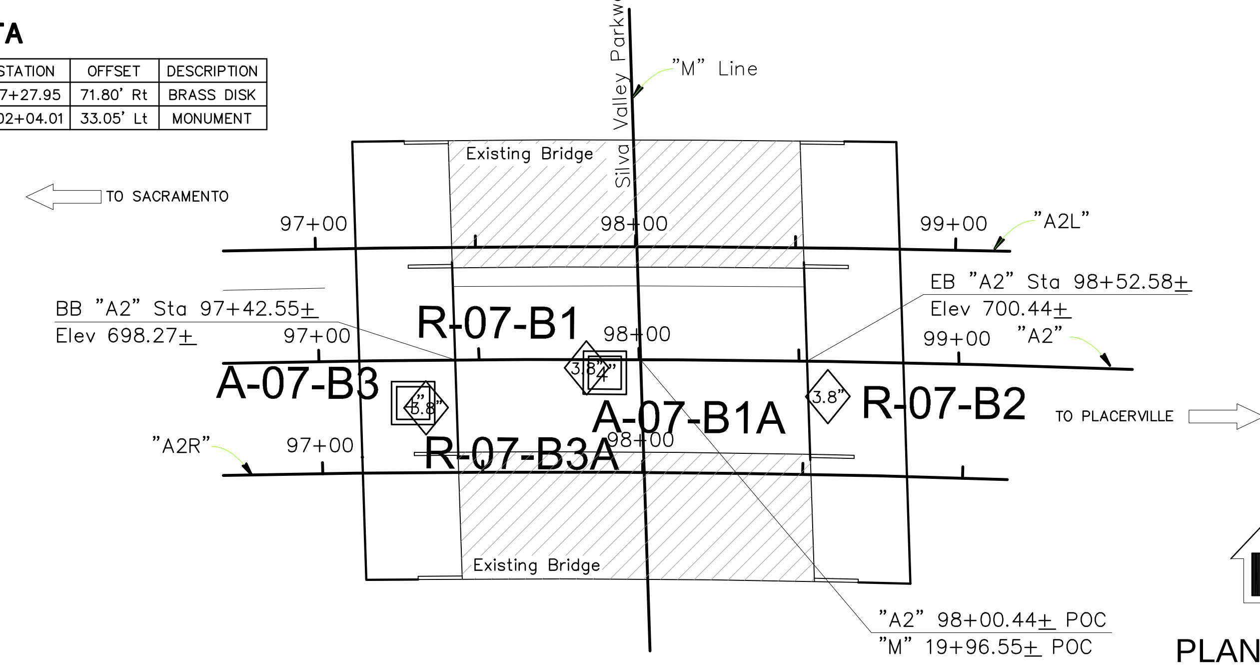
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 03252
EA 03-3A7111

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES 04/22/08
SHEET 14 OF 18

BENCH MARK DATA

NO.	NORTHING	EASTING	ELEVATION	LINE	STATION	OFFSET	DESCRIPTION
25124	2,002,545.43	6,830,886.32	695.47	"A2"	97+27.95	71.80' Rt	BRASS DISK
517	2,002,837.42	6,831,282.78	711.28	"A2L"	102+04.01	33.05' Lt	MONUMENT



PLAN
1" = 30'

NOTES:
1. Field classification of soil and rock was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2007). See Log of Test Borings 4 of 5 "Soil Legend" and Log of Test Borings 5 of 5 "Rock Legend".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a 140 lb. safety hammer operated with cat-head, rope and pully with a 30-inch drop. 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 but with brass liners.
4. 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.
5. 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 Caltrans Soil & Rock Logging, Classification, and Presentation Manual.
6. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 2007).
7. Where less than 0.5 feet of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
8. Consistency of soils shown in ( ) where estimated.
8. Ground water surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
9. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
10. Electronic media for preliminary plan view provided by Quincy Engineering, Inc., dated August 21, 2007.
11. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".
12. This LOTB was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	ED	50	0.0-2.9	448	451

CERTIFIED ENGINEERING GEOLOGIST — 08/11/08 DATE

PROFESSIONAL GEOLOGIST William E. Nichols No. 2229 Exp. 1-31-10 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

PLANS APPROVAL DATE

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

BLACKBURN CONSULTING
2437 FRONT STREET
WEST SACRAMENTO, CA 95691
File No. 1072.2

QUINCY ENGINEERING
3247 RAMOS CIRCLE
SACRAMENTO, CA 95827-2501

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	POST MILES-TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	ED	50	1.07/R2.40	301	349

CERTIFIED ENGINEERING GEOLOGIST — 05/16/12 DATE

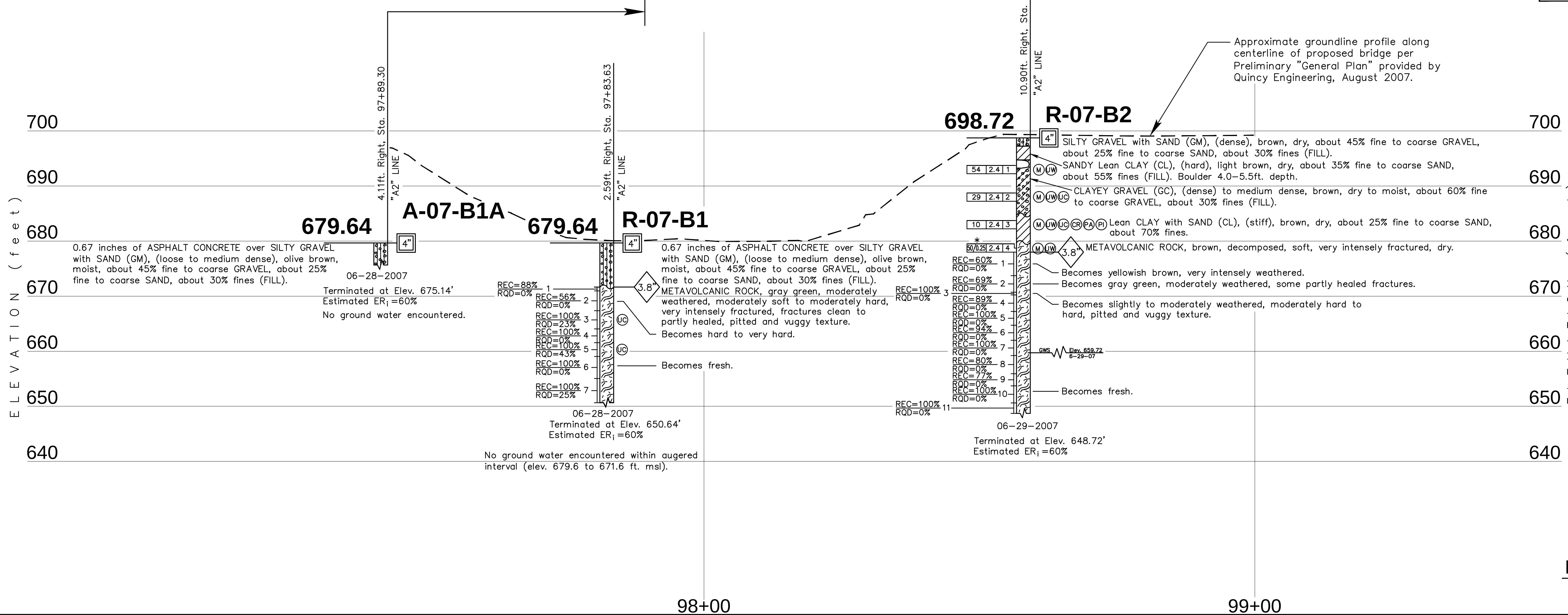
SILVA VALLEY WB ON-RAMP UC
LOG OF TEST BORINGS 6 OF 7

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA.
UNIT: 03-1E2901
PROJECT NUMBER & PHASE: 0300000258

Revisions shown on this Log of Test Borings are the addition of the following table and notes.

BRIDGE NO.	SHEET	OF
25-0129K	20	21

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



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

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DESIGN OVERSIGHT	BRIDGE NO. 25-0072	CLARKSVILLE UNDERCROSSING (WIDEN)
FUNCTIONAL SUPERVISOR	DRAWN BY: M. D. Robertson	FIELD INVESTIGATION BY: Rob Pickard, June 2007		NAME: Eric Fredrickson	POST MILE 1.66	LOG OF TEST BORINGS 2 of 5
NAME:	CHECKED BY: W. E. Nichols					

04/22/08 1072.2 Clarksville Road UC.dwg

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 03252
EA 03-3A7111

DISREGARD PRINTS BEARING EARLIER REVISION DATES — REVISION DATES 04/22/08 — SHEET 15 OF 18

APPENDIX C

Laboratory Test Results

- Silva Valley Westbound On-Ramp UC
- Clarksville UC (Widen)

Laboratory Test Results

Silva Valley Westbound On-Ramp UC

Blackburn Consulting
Silva Valley WB On-Ramp UC
File No. 556.2
November 2010

Laboratory Testing Summary

Exploration I.D.	Sample No.	Depth (feet)	Sample Type	USCS Classification	Moisture Content (%)	Dry Density, γ_{dry} (pcf)	Moisture Content (%)	Unconfined Compression (psi)	Corrosivity pH	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
R-10-005	S1	0.0-1.5	MC	ML					5.6	3220	13.6	35.5
R-10-005	Core	9.3-10.2	HQ	Rock				5,800				

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 09/17/2010
Date Submitted 09/14/2010

To: Ken Colburn
Blackburn Consulting
11521 Blocker Dr. Ste. 110
Auburn, CA 95603

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : SILVA VLY PKWY INTER Site ID : R-10-005-S1B.
Thank you for your business.

* For future reference to this analysis please use SUN # 58852-119539.

EVALUATION FOR SOIL CORROSION

Soil pH	5.63		
Minimum Resistivity	3.22	ohm-cm (x1000)	
Chloride	13.6 ppm	00.00136	%
Sulfate	35.5 ppm	00.00355	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Laboratory Test Results

Clarksville UC (Widen)

blackburn consulting

Project Name: Clarksville UC, El Dorado County, CA
BCI File No: 1072.1.A1.2
Date: 7/26/2007

Technician: MHW

MOISTURE-DENSITY TESTS

Sample No.	R-07-B2/1II	R-07-B2/4III	A-07-B3/1III	A-07-B3/2III			
Depth (ft.)	5.5-6.0	20.0-20.25	6.0-6.5	11.0-11.5			
Sample Length (in.)	5.80	5.03	5.74	5.54			
Diameter (in.)	2.43	2.40	2.40	2.43			
Sample Volume (ft^3)	0.01557	0.01317	0.01503	0.01487			
Tare No.	L	G	K	NN			
Tare (g)	191.7	198.7	212.9	104.3			
Wet Soil + Tare (g)	1144.8	918.7	1230.6	885.9			
Dry Soil + Tare (g)	1054.7	830.2	1124.4	829.6			
Dry Soil Weight (g)	863.1	631.5	911.5	725.3			
Water (g)	90.1	88.6	106.2	56.3			
Moisture (%)	**10.4**	**14.0**	**11.7**	**7.8**			
Dry Density (pcf)	**122.2**	**105.7**	**133.7**	**107.5**			

Sample: R-07-B2/1II Description: Olive brown silty sand to strong brown clayey silt (decomposed and weathered rock)
Moisture (Appearance): moist Consistency/Cementation:

Sample: R-07-B2/4III Description: Dark yellowish brown sandy clay to gravel (decomposed and weathered rock)
Moisture (Appearance): moist Consistency/Cementation:

Sample: A-07-B3/1III Description: Strong brown clayey sand (decomposed and weathered rock)
Moisture (Appearance): moist Consistency/Cementation:

Sample: A-07-B3/2III Description: Very dark greenish gray weathered rock and dark olive brown silty sand
Moisture (Appearance): moist Consistency/Cementation:

Sample: Description:
Moisture (Appearance): Consistency/Cementation:

Sample: Description:
Moisture (Appearance): moist Consistency/Cementation:

Sample: Description:
Moisture (Appearance): Consistency/Cementation:

Diameter = 1.44" for 1.5-inch Tubes
Diameter = 1.938" for 2-inch Tubes
Diameter = 2.438" for 2.5-inch Tubes
Diameter= 2.850" for 3.0-inch Shelby Tubes

Unconfined Compression Test Lab Sheet
ASTM D 2166-00

Project Name	Clarksville UC, El Dorado County, CA
Project Number	1072.1.A1.2
Sample	R-07-B2/2II Depth 10.5-11.0 ft
Sample Description	Dark red sandy lean clay (decomposed and weathered rock)
Date	7/26/2007
Tested By:	MHW

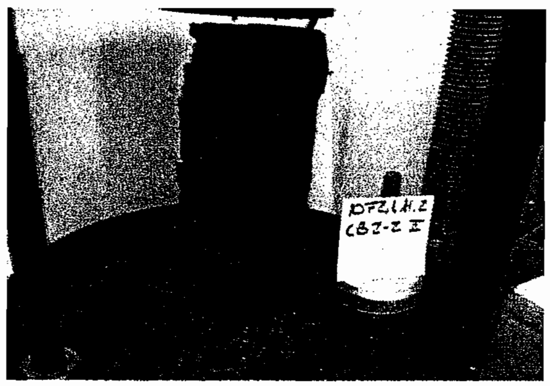
Original Sample Length	5.97
Original Diameter (in)	2.45
Sample Area (in^2)	4.71

axial strain	**4.5%**
Average cross-sectional area (in^2)	**4.94**
Average cross-sectional area (ft^2)	0.034
Peak Reading	0.630
Maximum Load(lb)	51
Compressive Strength (tsf)	**0.74**

Moisture Density

Wet Sample Weight (g)	1158.7
Tare Number	C
Tare Weight (g)	199.5
Dry Sample Weight (g)	1047.3
Dry Weight (g)	847.8
Water Weight (g)	111.4
Percent Moisture (%)	13.1
Wet Density (pcf)	129.8
Dry Density (pcf)	114.8

Remarks: * moisture taken after test



Compression Tests

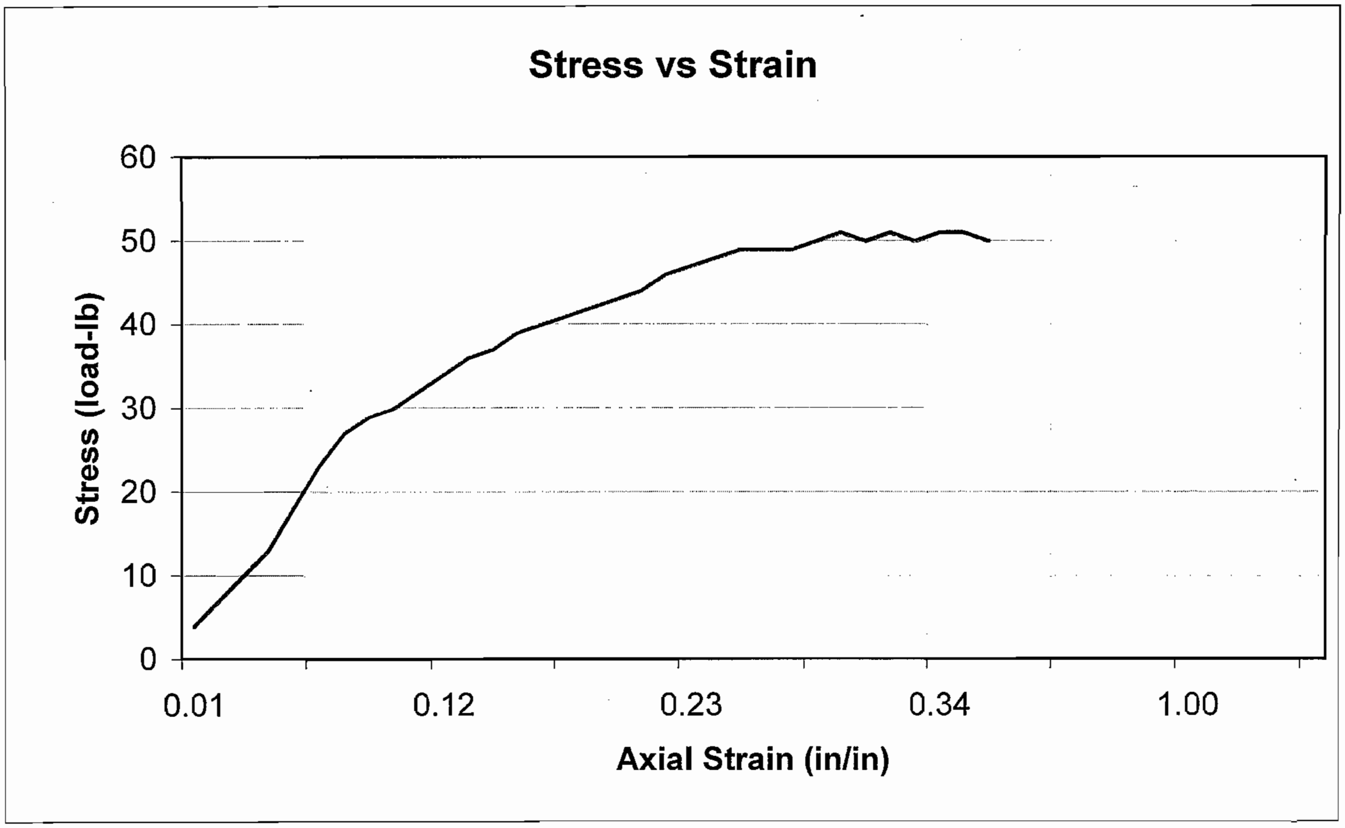
Dial reading @ 0 lb	0.900

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.890	4	0.730	42	0.570	50		
0.880	7	0.720	43				
0.870	10	0.710	44				
0.860	13	0.700	46				
0.850	18	0.690	47				
0.840	23	0.680	48				
0.830	27	0.670	49				
0.820	29	0.660	49				
0.810	30	0.650	49				
0.800	32	0.640	50				
0.790	34	0.630	51				
0.780	36	0.620	50				
0.770	37	0.610	51				
0.760	39	0.600	50				
0.750	40	0.590	51				
0.740	41	0.580	51				

Project
Clarksville UC, El Dorado County, CA
Project Number
1072.1.A1.2
Sample Number
R-07-B2/2II
Material Description
Dark red sandy lean clay (decomposed and weathered rock)
Tested By
MHW

ASTM D 2166-00



Wet Density (pcf)	129.8
Dry Density (pcf)	114.8
% Moisture	13.1

Unconfined Compressive Strength (tsf) 0.74

Unconfined Compression Test Lab Sheet
ASTM D 2166-00

Project Name	Clarksville UC, El Dorado County, CA
Project Number	1072.1.A1.2
Sample	R-07-B2/3II Depth 15.5-16.0 ft
Sample Description	Dark brown lean clay with sand
Date	7/26/2007
Tested By:	MHW

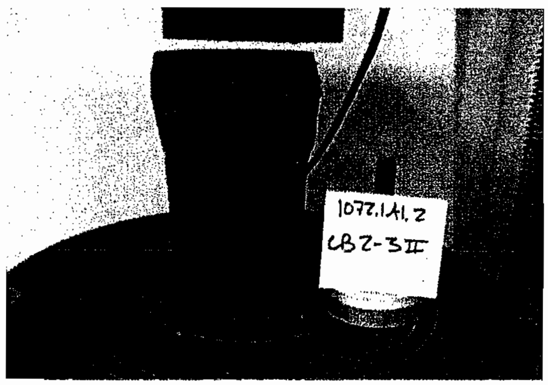
Original Sample Length	5.29
Original Diameter (in)	2.40
Sample Area (in^2)	4.52

axial strain	**10.8%**
Average cross-sectional area (in^2)	**5.07**
Average cross-sectional area (ft^2)	0.035
Peak Reading	0.330
Maximum Load(lb)	32
Compressive Strength (tsf)	**0.45**

Moisture Density

Wet Sample Weight (g)	919.3
Tare Number	QQ
Tare Weight (g)	104.9
Dry Sample Weight (g)	782.0
Dry Weight (g)	677.2
Water Weight (g)	137.3
Percent Moisture (%)	20.3
Wet Density (pcf)	129.6
Dry Density (pcf)	107.8

Remarks: * moisture taken after test



Compression Tests

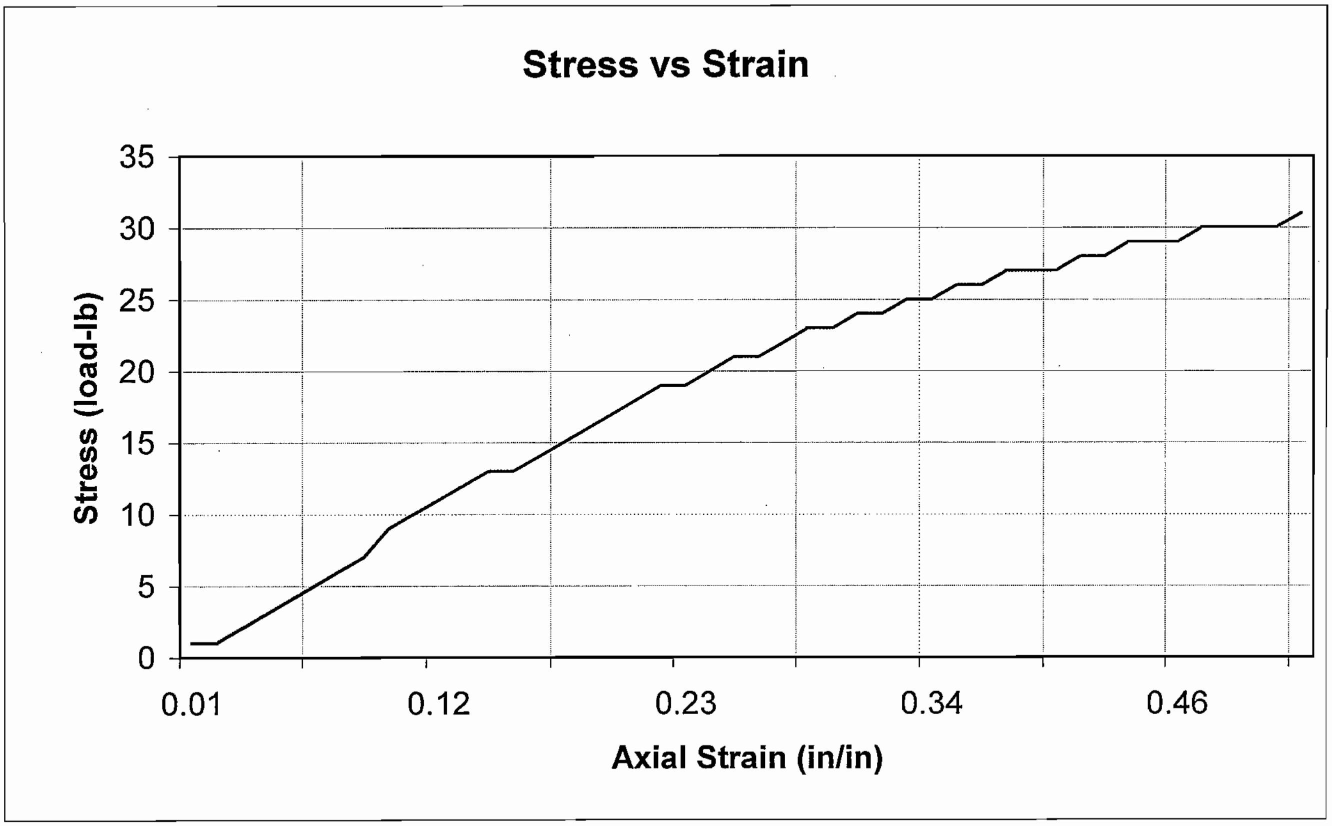
Dial reading @ 0 lb	0.900

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.890	1	0.730	16	0.570	26	0.410	31
0.880	1	0.720	17	0.560	27	0.400	31
0.870	2	0.710	18	0.550	27	0.390	31
0.860	3	0.700	19	0.540	27	0.380	31
0.850	4	0.690	19	0.530	28	0.370	31
0.840	5	0.680	20	0.520	28	0.360	31
0.830	6	0.670	21	0.510	29	0.350	31
0.820	7	0.660	21	0.500	29	0.340	31
0.810	9	0.650	22	0.490	29	0.330	32
0.800	10	0.640	23	0.480	30	0.320	32
0.790	11	0.630	23	0.470	30	0.310	32
0.780	12	0.620	24	0.460	30	0.300	32
0.770	13	0.610	24	0.450	30	0.290	32
0.760	13	0.600	25	0.440	31	0.280	32
0.750	14	0.590	25	0.430	31	0.270	32
0.740	15	0.580	26	0.420	31	0.260	32

Project
Clarksville UC, El Dorado County, CA
Project Number
1072.1.A1.2
Sample Number
R-07-B2/3II
Material Description
Dark brown lean clay with sand
Tested By
MHW

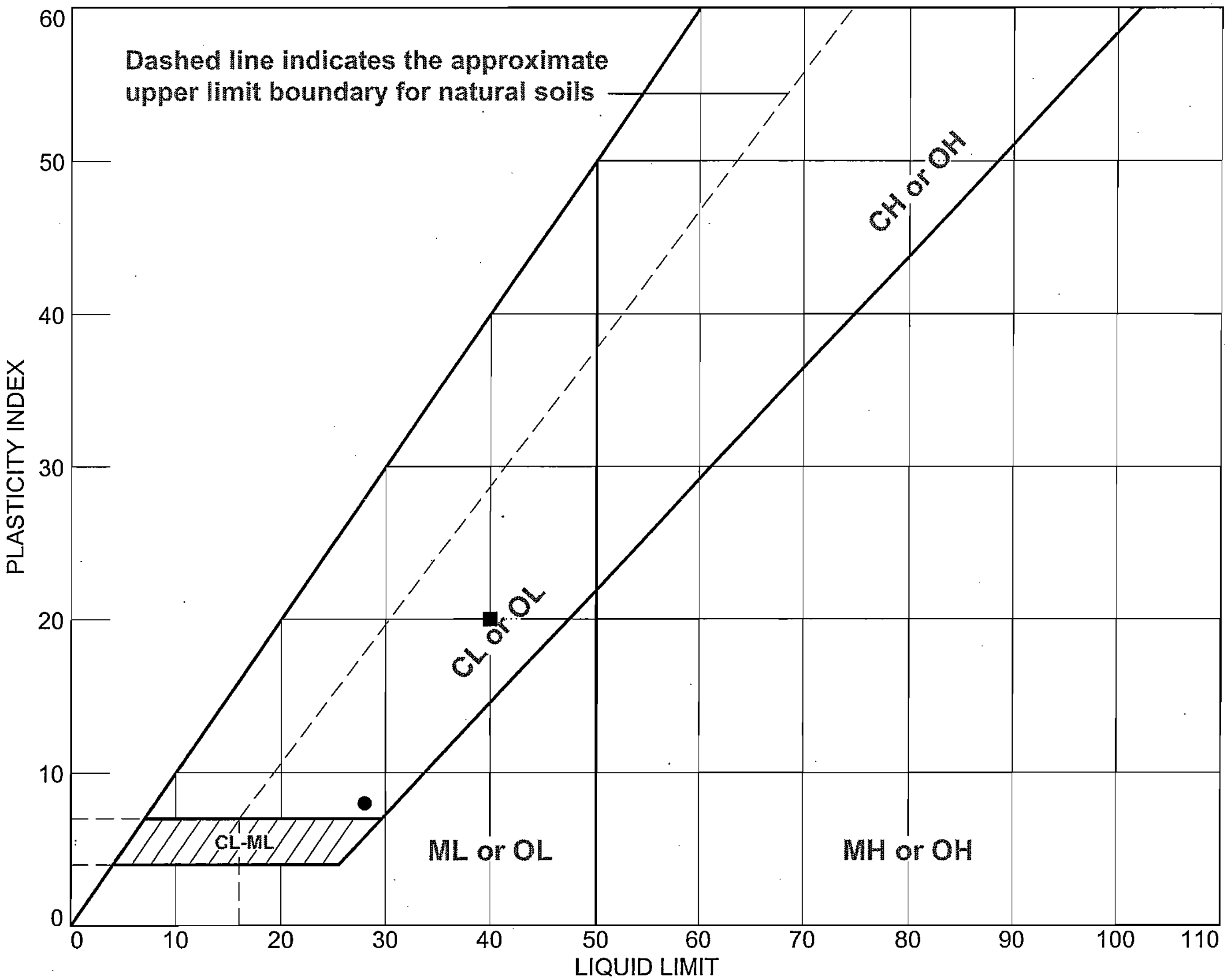
ASTM D 2166-00



Wet Density (pcf)	129.6
Dry Density (pcf)	107.8
% Moisture	20.3

Unconfined Compressive Strength (tsf) 0.45

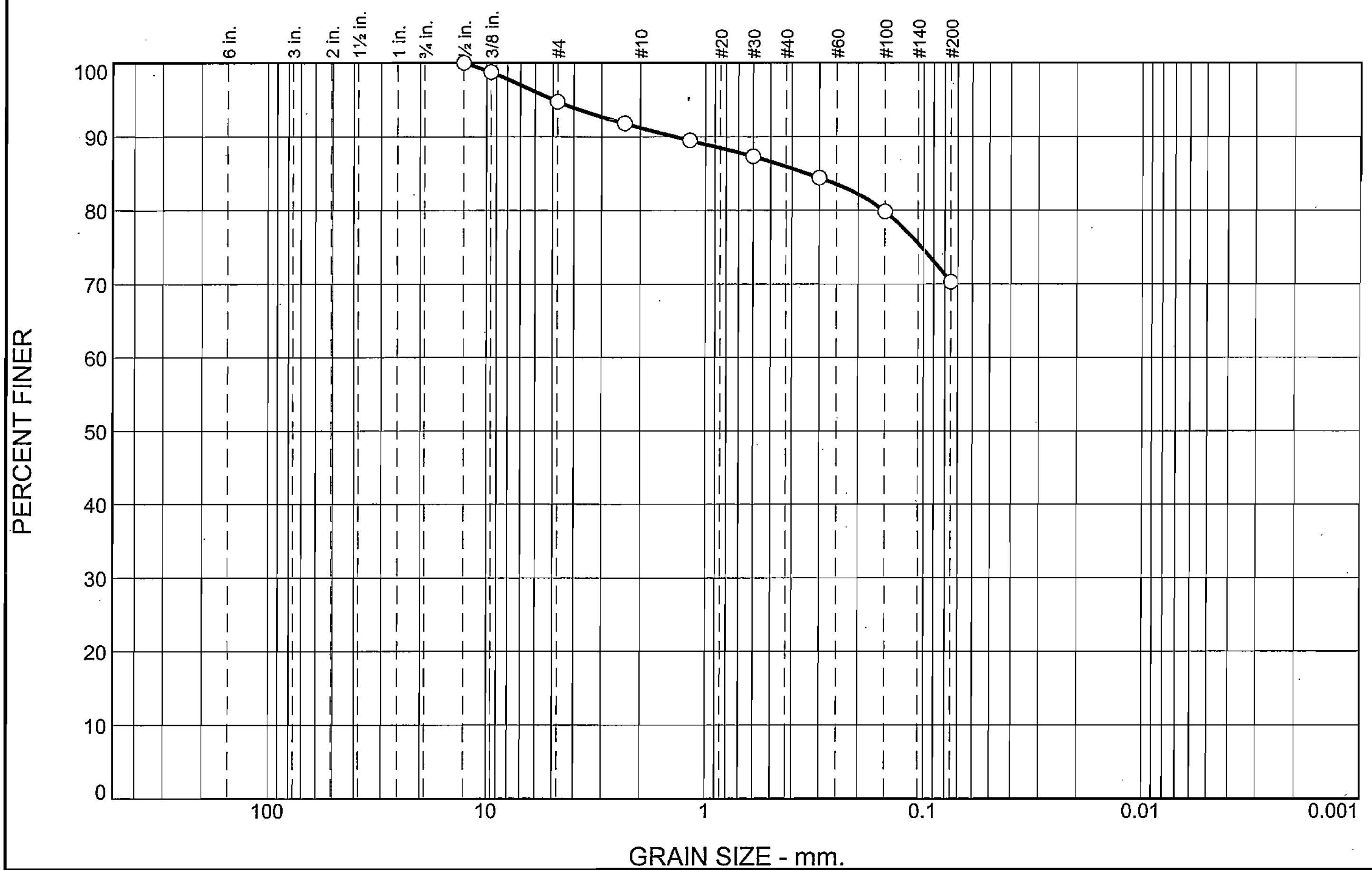
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		R-07-B2/3 III	16.0-16.5 ft		20	28	8	CL
■		R-07-B3A/ Run 3	15.0-19.5 ft		20	40	20	SC

Blackburn Consulting **W. Sacramento, CA**	**Client:** Quincy Engineering, Inc. **Project:** Clarksville UC, El Dorado County, CA **Project No.:** 1072.1.A1.2 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.3	3.5	5.2	15.7	70.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	98.8		
#4	94.7		
#8	91.8		
#16	89.5		
#30	87.3		
#50	84.4		
#100	79.8		
#200	70.3		

* (no specification provided)

Material Description

Very dark brown sandy lean clay

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 0.3400 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(4)

Remarks

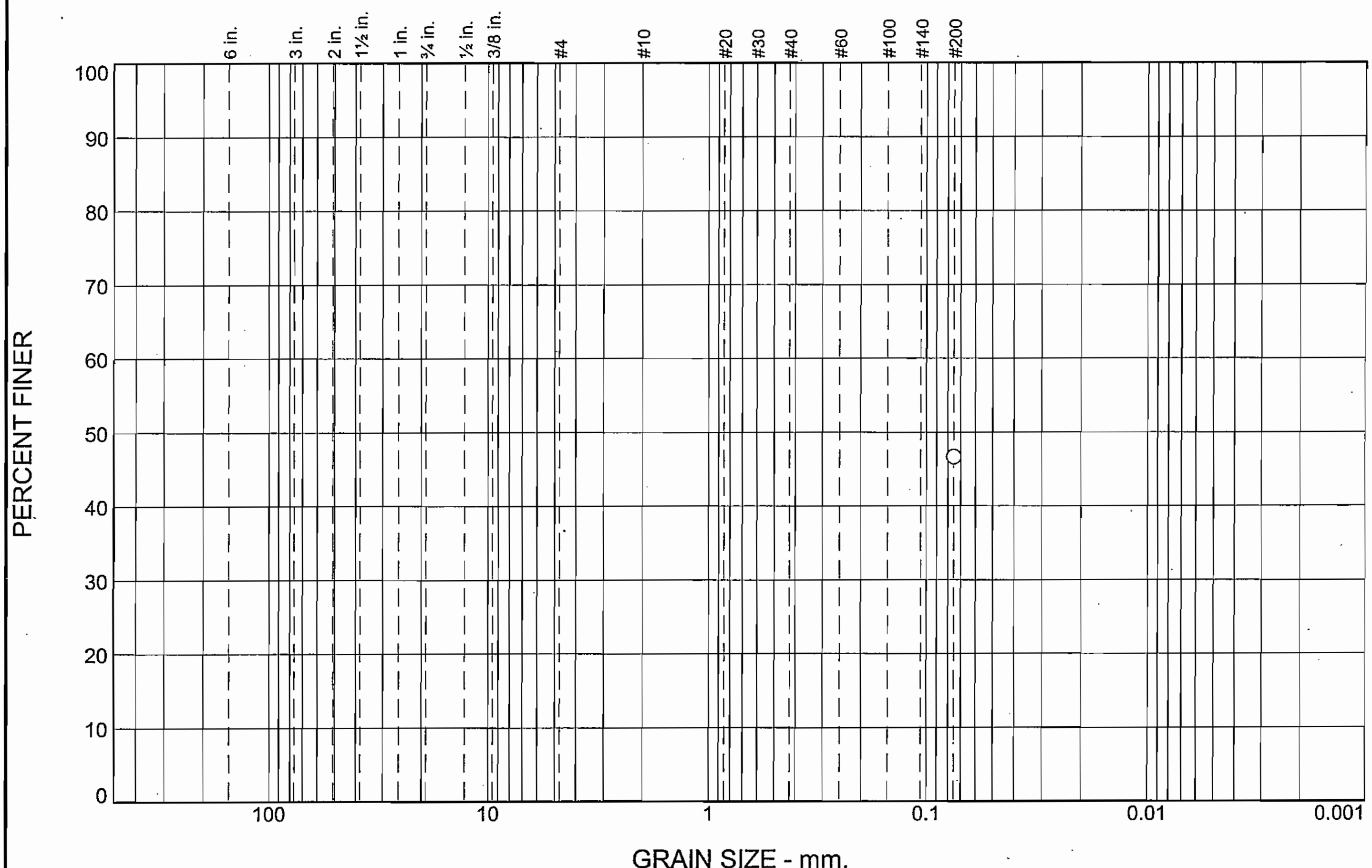
Sample Number: R-07-B2/3 III **Depth:** 16.0-16.5 ft **Date:** 7-27-07

Blackburn Consulting

W. Sacramento, CA

Client: Quincy Engineering, Inc.
Project: Clarksville UC, El Dorado County, CA
Project No: 1072.1.A1.2 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						46.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	46.7		

* (no specification provided)

Material Description

Very dark grayish brown clayey sand

Atterberg Limits

PL= 20 LL= 40 PI= 20

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample Number: R-07-B3A/Run 3 **Depth:** 15.0-19.5 ft **Date:** 7-27-07

Blackburn Consulting

W. Sacramento, CA

Client: Quincy Engineering, Inc.
Project: Clarksville UC, El Dorado County, CA

Project No: 1072.1.A1.2 **Figure**

blackburn consulting

2437 Front Street
West Sacramento, CA 95691
Phone: 916.375.8706
Fax: 916.375.8709

Project: Clarksville UC (Widen)
File No.: 1072.1

UNCONFINED COMPRESSIVE STRENGTH TEST REPORT

ASTM D 2938-95

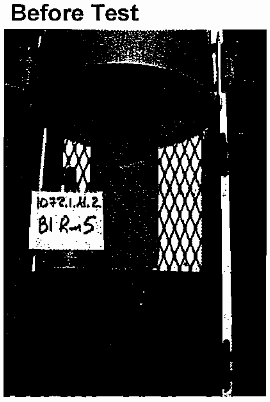
Sample ID	Description	Length (inches)^3	Dia. (inches)	Area (in^2)	Moisture (%)	Temp. (°f)	Max Load (psf)^2	Strength (tsf)
R-07-B1/Run 3 (12.3-15.6)	Metavolcanic Rock, gray green, slightly weathered, very hard, very intensely fractured	5.07	2.38	4.45	n/a	75	35704	575.9

Before Test

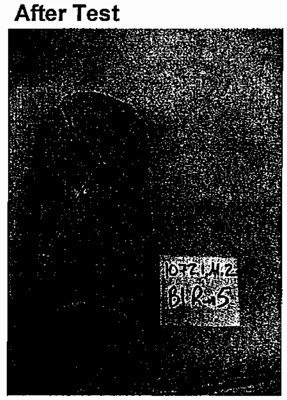
After Test

Sample ID	Description	Length (inches)^3	Dia. (inches)	Area (in^2)	Moisture (%)	Temp. (°f)	Max Load (psf)^2	Strength (tsf)
R-07-B1/Run 5 (18.1-20.6)	Metavolcanic Rock, gray green, slightly weathered to fresh, very hard, very intensely fractured	5.00	2.38	4.45	n/a	75	8251	133.1

Before Test



After Test



NOTES:
1. Rate of Strain=0.50in./inch using a Humboldt "Master Loader", 10,000 lb. maximum capacity.
2. Rate of Strain=10,000lbs./min. using a Forney Press, 100,000 lb. capacity.
3. Cores cut using a wet saw with a diamond blade.

blackburn consulting
2437 Front Street
West Sacramento, CA 95691
Phone: 916.375.8706
Fax: 916.375.8709

Project: Clarksville UC (Widen)
File No.: 1072.1

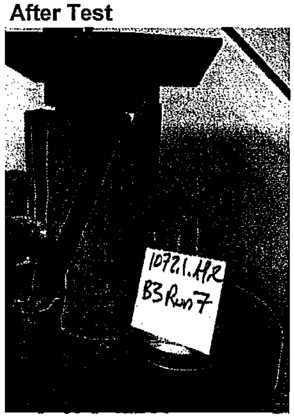
UNCONFINED COMPRESSIVE STRENGTH TEST REPORT

ASTM D 2938-95

Sample ID	Description	Length (inches)[3]	Dia. (inches)	Area (in^2)	Moisture (%)	Temp. (°f)	Max Load (psf)[1]	Strength (tsf)
R-07-B3A/Run 7 (28.0-30.0)	Metavolcanic Rock, gray green, slightly weathered, hard, very intensely fractured	5.34	2.39	4.49	n/a	75	1531	24.7

Before Test

After Test

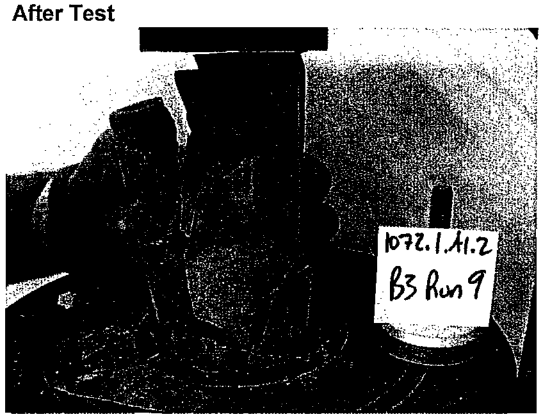


NOTES:
Sample sheared on fracture plane

Sample ID	Description	Length (inches)[3]	Dia. (inches)	Area (in^2)	Moisture (%)	Temp. (°f)	Max Load (psf)[1]	Strength (tsf)
R-07-B3A/Run 9 (32.5-35.5)	Metamorpic Rock, gray green, slightly weathered to fresh, hard, very intensely fractured	5.23	2.39	4.49	n/a	75	8800	141.9

Before Test

After Test



NOTES:
1. Rate of Strain=0.50in./inch using a Humboldt "Master Loader", 10,000 lb. maximum capacity.
2. Rate of Strain=10,000lbs./min. using a Forney Press, 100,000 lb. capacity.
3. Cores cut using a wet saw with a diamond blade.

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 08/01/2007
Date Submitted 07/25/2007

To: Nikki Hart
Blackburn Consulting
2437 Front Street
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : ~~LATROBE~~/CLARKSVL UC Site ID : B2-3 III.
Thank you for your business.

* For future reference to this analysis please use SUN # 51268-102390.

EVALUATION FOR SOIL CORROSION

Soil pH	6.02		
Minimum Resistivity	2.68	ohm-cm (x1000)	
Chloride	9.1 ppm	00.00091	%
Sulfate	17.0 ppm	00.00170	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical

11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 08/01/2007
Date Submitted 07/25/2007

To: Nikki Hart
Blackburn Consulting
2437 Front Street
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : ~~LATROBE~~/CLARKSVL UC Site ID : B3A RUN 3.
Thank you for your business.

* For future reference to this analysis please use SUN # 51268-102391.

EVALUATION FOR SOIL CORROSION

Soil pH	6.49	
Minimum Resistivity	0.80 ohm-cm (x1000)	
Chloride	9.8 ppm	00.00098 %
Sulfate	274.6 ppm	00.02746 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX D

Design Calculations

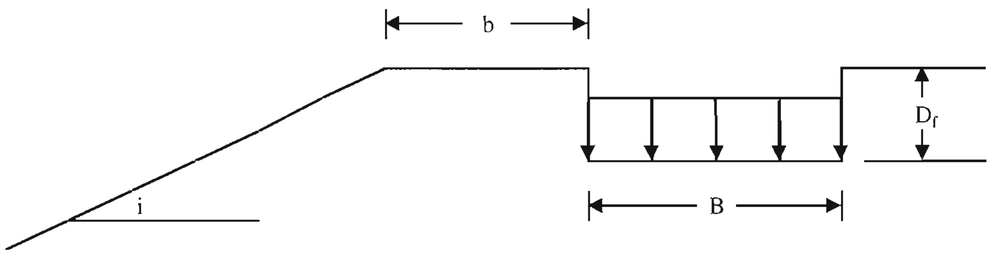
- Allowable Bearing Capacity and Settlement – Abutments
- Allowable Bearing Capacity and Settlement – Bents
- Elastic Constants of Various Soils
- Slope Stability Output Graphs
- Lateral Earth Pressure

Design Calculations

Allowable Bearing Capacity and Settlement – Abutments

Modified Bearing Capacity Factor for Footing Adjacent to Sloping Ground after Meyerhof (1957)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
Support: Abutment 1 --Footing Established on 4 ft thick fill prism
Boring: R-07-B3A, Clarksville UC (Widen)
BCI No.: 556.2
By: WEN



Input Parameters:

Depth to Bottom of Footing, D_f = 4.90 feet
Footing Width, B = 7.20 feet
Footing to Slope Distance, b = 0.00 feet
Slope Inclination, i = 26.6 degrees
D_f/B = 0.68 ($D/B \leq 1$)
b/B = 0.00

Soil Unit Weight, γ = 125 (pcf)
Friction Angle, ϕ = 34 ($\phi \geq 30°$)
Cohesion, c = 0 (psf)

By Interpolation:

At D_f/B = 0

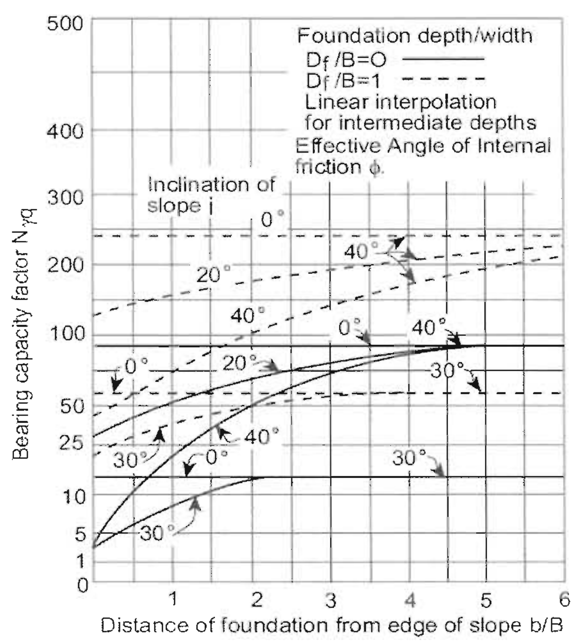
ϕ	$N\gamma q$
30	3.5
34	11.0
40	22.4

At D_f/B = 1

ϕ	$N\gamma q$
30	26.9
34	54.2
40	95.3

D_f/B	$N\gamma q$
0.00	11.0
0.68	**40.4**
1.00	54.2

$N\gamma q$ = **40.4** (Modified Bearing Capacity Factor)



Cohesionless Soil

Reference: AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Figure 10.6.3.1.2c-2 Modified Bearing Capacity Factors for Footing in Cohesionless Soils and Adjacent to Sloping Ground after Meyerhof (1957).

Allowable Bearing and Immediate Settlement Worksheet (WSD)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
BCI No: 556.2

Support: Abutment 1 --Footing Established on 4 ft thick fill prism
Boring: R-07-B3A, Clarksville UC (Widen)

LRFD Service Limit State I Vertical Load (kips):	760	
Effective Footing Width, B'_f (feet):	7.20	
Effective Footing Length, L'_f (feet):	40.90	
Ground Surface Elevation (feet):	689.5	(equal to footing bottom for a footing in fill above ex. grnd. surface)
Ground Water Elevation (feet):	684.0	
Depth to Ground Water (feet):	5.5	(for settlement analysis)
Depth of footing (feet):	0.0	
Time to Settlement (t):	1.2	
Bottom Footing Elevation (feet):	689.5	
Finished Grade (feet):	694.0	
Depth to Ground Water (feet):	10.0	(for bearing resistance analysis)
Depth of footing (feet):	4.5	
γ (pcf) =	125	Soil Parameters at base of footing
ϕ (degrees) =	34	
c (psf) =	0	
Factor of Safety =	3.0	

Layer	Material Description	Depth Bottom Layer (feet)	Layer Thickness (feet)	Top Elev. (feet)	Bottom Elev. (feet)	Soil Unit Weight (pcf)	Soil Type (1, 2, 3, or 4)	$N1_{60}$	Es (tsf)	or	Estimated Es (tsf)
1	Str. Backfill	4.0	4.0	689.5	685.5	125	3	16	160		
2	Residual Soil	6.5	2.5	685.5	683.0	115	1	29	116		
3	Int. Wthd Rock	11.5	5.0	683.0	678.0	125	3	70	700		
4	Wthd Rock	38.0	26.5	678.0	651.5	130	4				2000
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Soil Types
1) Silts, sandy silts, slightly cohesive mixtures
2) Clean fine to medium sands and slightly silty sands
3) Coarse sands and sands with little gravel
4) Sandy gravel and gravels

Ulimate Gross Bearing Capacity q_{ult} (ksf)	Allowable Gross Bearing Capacity q_{all} (ksf)
13.73	4.58

Gross Uniform Bearing Stress q_o (ksf)	Net Bearing Stress q'_o (ksf)	Immediate Settlement S_i (inches)
2.58	2.58	0.35

Sevice Limit State Settlement (1.0 inches) Check		
q_o (ksf)	<	q_{pg} (ksf)
2.58		6.40
OKAY		

Permissible Net Contact Stress q_{pn} (ksf)	Permissible Gross Contact Stress q_{pg} (ksf)	Immediate Settlement S_i (inches)
6.40	6.40	1.00

Sevice Limit State Bearing Capacity Check		
q_o (ksf)	<	q_{all} (ksf)
2.58		4.58
OKAY		

References

1) Caltrans, Memo To Designers 4-1 Spread Footings, April 2008.
2) Nominal Bearing Resistance Equation (10.6.3.1.2a-1) Modified for Footing Near Slope, AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.
3) Schmertmann's Modified Method for Calculation of Immediate Settlements (1978), Soils and Foundations - Volume II, FHWA NHI-06-089, December 2006.
4) Elastic Constants of Various Soils (Table C10.4.6.3-1) AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

BEARING CAPACITY for FOOTING LOCATED ADJACENT to SLOPING GROUND

STRENGTH LIMIT STATE (AASHTO Bridge Design Specifications)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
BCI No: 556.2

Support: Abutment 1 --Footing Established on 4 ft thick fill prism
Boring: R-07-B3A, Clarksville UC (Widen)

Equation: $q_n = cN_{cqm} + 0.5\,\gamma B N_{\gamma qm} C_{w\gamma}$

in which:

$N_{cqm} = N_{cq} s_c i_c$

$N_{\gamma qm} = N_{\gamma q} s_\gamma i_\gamma$

D_w	$C_{w\gamma}$
0	0.5
D_f	0.5
>1.5B+D_f	1.0

where:

q_n = nominal bearing resistance
c = cohesion (psf)
B' = effective footing width (feet)
γ = total (moist) unit weight of soil (pcf)
D_f = footing embedment depth (feet)
N_{cq} *and* $N_{\gamma q}$ = modified bearing capacity factors
$C_{w\gamma}$ = correction factors for location of ground water
s_c *and* s_γ = footing shape correction factors
i_c *and* i_γ = load inclination factors
D_w = depth to ground water taken from the ground surface (feet)

Input Parameters

γ =	125	(pcf)
ϕ =	34	(degrees)
c =	0	(psf)
D_f =	4.5	(feet)
D_w =	10	(feet)

i_c =	1.0
i_γ =	1.0

Bottom Footing Elevation (feet):	689.5
Finished Grade (feet):	694.0
Ground Water Elevation (feet):	684.0

Strength Limit State

Solve for Ultimate Gross Bearing Capacity

Effective Footing Dimensions B'	L'		$C_{w\gamma}$	s_c	s_γ		**Ulimate Gross Bearing Capacity** (psf)	(ksf)	(tsf)	**Allowable Gross Bearing Capacity** Factor of Safety = 3.0 (psf)	(ksf)	(tsf)
(feet)												
7.2	40.9		0.75	1.00	1.00		13733	13.73	6.9	4578	4.58	2.3

Modified Bearing Capacity Factors

N_{cq} =	NA
$N_{\gamma q}$ =	40.4

Shape Correction Factors

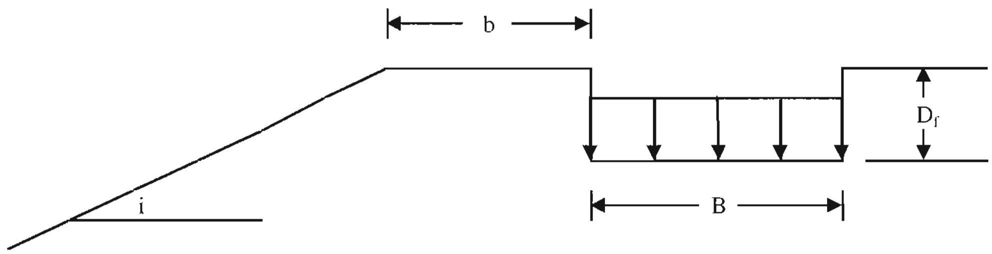
ϕ	s_c	s_γ
$\phi = 0$	1 + (B/5L)	1.0
$\phi > 0$	1	1 - 0.4(B/L)

Notes: If L > 5B, then s_c and s_γ= 1.0 (Geotechnical Engineering Circular No. 6, FHWA-SA-02-054, pgs 55-56)

$N\gamma q$ determined from Figure 10.6.3.1.2c-2, AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Modified Bearing Capacity Factor for Footing Adjacent to Sloping Ground after Meyerhof (1957)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
Support: Abutment 4 -- Footing Established on 4 ft thick fill prism
Boring: R-10-005
BCI No.: 556.2
By: WEN



Input Parameters:

Depth to Bottom of Footing, D_f = 4.90 feet
Footing Width, B = 7.40 feet
Footing to Slope Distance, b = 0.00 feet
Slope Inclination, i = 26.6 degrees
D_f/B = 0.66 ($D/B \leq 1$)
b/B = 0.00

Soil Unit Weight, γ = 125 (pcf)
Friction Angle, ϕ = 34 ($\phi \geq 30°$)
Cohesion, c = 0 (psf)

By Interpolation:

At D_f/B = 0

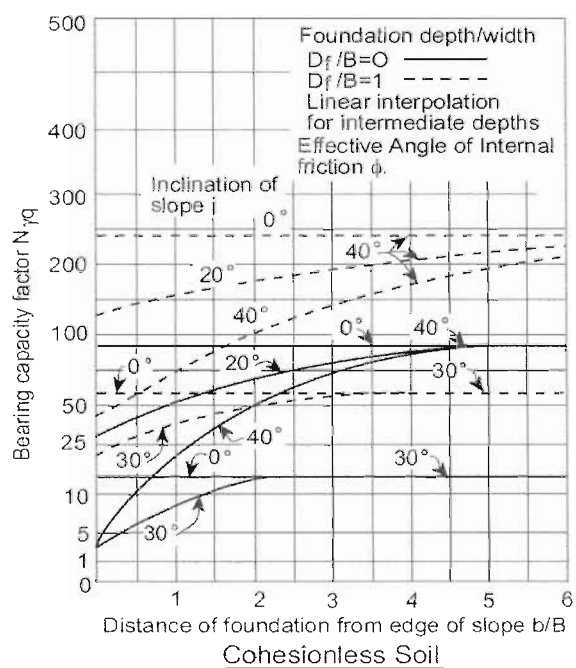
ϕ	$N\gamma q$
30	3.5
34	11.0
40	22.4

At D_f/B = 1

ϕ	$N\gamma q$
30	26.9
34	54.2
40	95.3

D_f/B	$N\gamma q$
0.00	11.0
0.66	**39.6**
1.00	54.2

$N\gamma q$ = **39.6** (Modified Bearing Capacity Factor)



Reference: AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

Figure 10.6.3.1.2c-2 Modified Bearing Capacity Factors for Footing in Cohesionless Soils and Adjacent to Sloping Ground after Meyerhof (1957).

Allowable Bearing and Immediate Settlement Worksheet (WSD)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
BCI No: 556.2

Support: Abutment 4 -- Footing Established on 4 ft thick fill prism
Boring: R-10-005

LRFD Service Limit State I Vertical Load (kips):	820	
Effective Footing Width, B'_f (feet):	7.40	
Effective Footing Length, L'_f (feet):	40.90	
Ground Surface Elevation (feet):	692.5	(equal to footing bottom for a footing in fill above ex. grnd. surface)
Ground Water Elevation (feet):	684.0	
Depth to Ground Water (feet):	8.5	(for settlement analysis)
Depth of footing (feet):	0.0	
Time to Settlement (t):	1.2	
Bottom Footing Elevation (feet):	692.5	
Finished Grade (feet):	697.4	
Depth to Ground Water (feet):	13.4	(for bearing resistance analysis)
Depth of footing (feet):	4.9	
γ (pcf) =	125	Soil Parameters at base of footing
ϕ (degrees) =	34	
c (psf) =	0	
Factor of Safety =	3.0	

Layer	Material Description	Depth Bottom Layer (feet)	Layer Thickness (feet)	Top Elev. (feet)	Bottom Elev. (feet)	Soil Unit Weight (pcf)	Soil Type (1, 2, 3, or 4)	$N1_{60}$	Es (tsf)	or	Estimated Es (tsf)
1	Str. Backfill	4.0	4	692.5	688.5	125	3	16	160		
2	Residual Soil	5.5	1.5	688.5	687.0	115	1	29	116		
3	Int. Wthd Rock	9.5	4	687.0	683.0	125	3	70	700		
4	Wthd Rock	29.0	19.5	683.0	663.5	130	4				2000
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Soil Types

1) Silts, sandy silts, slightly cohesive mixtures
2) Clean fine to medium sands and slightly silty sands
3) Coarse sands and sands with little gravel
4) Sandy gravel and gravels

Ulimate Gross Bearing Capacity q_{ult} (ksf)	Allowable Gross Bearing Capacity q_{all} (ksf)
16.19	5.40

Gross Uniform Bearing Stress q_o (ksf)	Net Bearing Stress q'_o (ksf)	Immediate Settlement S_i (inches)
2.71	2.71	0.30

Sevice Limit State Settlement (1.0 inch) Check		
q_o (ksf)	<	q_{pg} (ksf)
2.71		7.70
OKAY		

Permissible Net Contact Stress q_{pn} (ksf)	Permissible Gross Contact Stress q_{pg} (ksf)	Immediate Settlement S_i (inches)
7.70	7.70	1.00

Sevice Limit State Bearing Capacity Check		
q_o (ksf)	<	q_{all} (ksf)
2.71		5.40
OKAY		

References

1) Caltrans, Memo To Designers 4-1 Spread Footings, April 2008.
2) Nominal Bearing Resistance Equation (10.6.3.1.2a-1) Modified for Footing Near Slope, AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.
3) Schmertmann's Modified Method for Calculation of Immediate Settlements (1978), Soils and Foundations - Volume II, FHWA NHI-06-089, December 2006.
4) Elastic Constants of Various Soils (Table C10.4.6.3-1) AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

BEARING CAPACITY for FOOTING LOCATED ADJACENT to SLOPING GROUND

STRENGTH LIMIT STATE (AASHTO Bridge Design Specifications)

Date: 5/8/2012
Project: Silva Valley Westbound On-Ramp UC
BCI No: 556.2

Support: Abutment 4 -- Footing Established on 4 ft thick fill prism
Boring: R-10-005

Equation: $q_n = cN_{cqm} + 0.5\,\gamma\,BN_{\gamma qm}\,C_{w\gamma}$

in which:

$N_{cqm} = N_{cq}\,s_c\,i_c$

$N_{\gamma qm} = N_{\gamma q}\,s_\gamma\,i_\gamma$

D_w	$C_{w\gamma}$
0	0.5
D_f	0.5
$>1.5B+D_f$	1.0

where:

q_n = nominal bearing resistance
c = cohesion (psf)
B' = effective footing width (feet)
γ = total (moist) unit weight of soil (pcf)
D_f = footing embedment depth (feet)
N_{cq} *and* $N_{\gamma q}$ = modified bearing capacity factors
$C_{w\gamma}$ = correction factors for location of ground water
s_c *and* s_γ = footing shape correction factors
i_c *and* i_γ = load inclination factors
D_w = depth to ground water taken from the ground surface (feet)

Input Parameters

γ =	125	(pcf)
ϕ =	34	(degrees)
c =	0	(psf)
D_f =	4.9	(feet)
D_w =	13.4	(feet)

i_c =	1.0
i_γ =	1.0

Bottom Footing Elevation (feet):	692.5
Finished Grade (feet):	697.4
Ground Water Elevation (feet):	684.0

Solve for Ultimate Gross Bearing Capacity

Effective Footing Dimensions							Ulimate Gross Bearing Capacity			Strength Limit State — Allowable Gross Bearing Capacity (Factor of Safety = 3.0)		
B'	L'		$C_{w\gamma}$	s_c	s_γ							
(feet)							(psf)	(ksf)	(tsf)	(psf)	(ksf)	(tsf)
7.4	40.9		0.88	1.00	1.00		16189	16.19	8.1	5396	5.40	2.7

Modified Bearing Capacity Factors

N_{cq} =	NA
$N_{\gamma q}$ =	39.6

Shape Correction Factors

ϕ	s_c	s_γ
$\phi = 0$	1 + (B/5L)	1.0
$\phi > 0$	1	1 - 0.4(B/L)

Notes: If L > 5B, then s_c and s_γ = 1.0 (Geotechnical Engineering Circular No. 6, FHWA-SA-02-054, pgs 55-56)

$N\gamma q$ determined from Figure 10.6.3.1.2c-2, AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

ELASTIC CONSTANTS OF VARIOUS SOILS
(after AASHTO LRFD BDS)

Typical Range of Values

Soil Type	Young's Modulus, Es (tsf)	Poisson's Ratio, ν (dim)
Clay:		
Soft sensitive	25-150	
Medium stiff to stiff	150-500	0.4-0.5 (undrained)
Very stiff	500-1000	
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine sand:		
Loose	80-120	
Medium dense	120-200	0.25
Dense	200-300	
Sand:		
Loose	100-300	0.2-0.35
Medium dense	300-500	
Dense	500-800	0.3-0.4
Gravel:		
Loose	300-800	0.2-0.35
Medium dense	800-1000	
Dense	1000-2000	0.3-0.4

Estimating Es from SPT N-value ($N1_{60}$)(1)

Soil Type	Young's Modulus, Es (tsf)
1) Silts, sandy silts, slightly cohesive mixtures	$4N1_{60}$
2) Clean fine to medium sands and slightly silty sands	$7N1_{60}$
3) Coarse sands and sands with little gravel	$10N1_{60}$
4) Sandy gravel and gravels	$12N1_{60}$

ELASTIC CONSTANTS OF VARIOUS SOILS
MODIFIED AFTER US DEPARTMENT OF THE NAVY (1982) AND BOWLES (1982)

Estimating Es from S_u(2)

Soil Type	Young's Modulus, Es (tsf)
Soft sensitive clay	$200S_u$-$500S_u$
Medium stiff to stiff clay	$750S_u$-$1,200S_u$
Very stiff clay	$1,500S_u$-$2,000S_u$

(1) $N1_{60}$ = SPT corrected for depth and overburden.
(2) S_u = Undrained shear strength (tsf).

Sources: Typical Ranges of Values / Estimating Es from SPT N-value
Table C10.4.6.3-1, AASHTO LRFD Bridge Design Specifications, 4th Edition.
Table 5-16, FHWA NHI-06-088, Soils and Foundations - Volume 1, December 2006.
Estimating Es from S_u
Caltrans Bridge Design Specifications, November 2003.

Design Calculations

Allowable Bearing Capacity and Settlement – Bents

Nominal Bearing Resistance and Immediate Settlement Worksheet (LRFD)

Date:	3/19/2012	*Support:*	BENT 2
Project:	SILVA VALLEY WB ON-RAMP UC	*Boring:*	R-10-005
BCI No:	556.2	*By:*	WEN
		Check by:	*Date:*

		Eff. Footing Width, B'f (feet)	Eff. Footing Length, L'f (feet)
LRFD Service Limit State I Vertical Load (kips):	1050	7.2	39.4
LRFD Strength Limit State Load (kips):	1440	6.6	38.0
LRFD Extreme Event Limit State Load (kips):	820	8.3	42.0

Ground Surface Elevation (feet):	684.0	(equal to footing bottom for a footing in fill above ex. grnd. surface)
Ground Water Elevation (feet):	684.0	
Depth to Ground Water (feet):	0.0	(for settlement analysis)
Depth of footing (feet):	6.5	
Time to Settlement (t):	1.2	
Bottom Footing Elevation (feet):	677.5	
Finished Grade (feet):	684.0	
Depth to Ground Water (feet):	0.0	(for bearing resistance analysis)
Depth of footing (feet):	6.5	
γ (pcf) =	130	Soil Parameters at base of footing
ϕ (degrees) =	38	
c (psf) =	0	
Resistance Factor (φ_b)=	0.45	

Layer	Soil Description	Depth Bottom Layer (feet)	Layer Thickness (feet)	Top Elev. (feet)	Bottom Elev. (feet)	Soil Unit Weight (pcf)	Soil Type (1, 2, 3, or 4)	$N1_{60}$	Es (tsf)	or	Estimated Es (tsf)
1	Int Wthd Rock	5.5	5.5	684.0	678.5	125	3	70	700		
2	Wthd Rock	30	24.5	678.5	654.0	130	4				2000
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Soil Types
1) Silts, sandy silts, slightly cohesive mixtures
2) Clean fine to medium sands and slightly silty sands
3) Coarse sands and sands with little gravel
4) Sandy gravel and gravels

Gross Nominal Bearing Resistance q_n (ksf)	Factored Gross Nominal Bearing Resistance q_R (ksf)
37.41	16.83

Gross Uniform Bearing Stress (Service Limit) q_o (ksf)	Net Bearing Stress q'o (ksf)	Gross Uniform Bearing Stress (Strength Limit) q_o (ksf)	Gross Uniform Bearing Stress (Extreme Limit) q_o (ksf)
3.70	3.29	5.74	2.35

Sevice Limit State Settlement (1 inch) Check		
q'_o (ksf)	<	q_{pn} (ksf)
3.29		25.00
OKAY		

Permissible Net Contact Stress q_{pn} (ksf)	Permissible Gross Contact Stress q_{pg} (ksf)	Immediate Settlement S_i (inches)
25.00	25.41	1.00

Immediate Settlement under Net Bearing Stress due to Service Limit I State Load Combination S_i, (inches)
0.08

The Net Bearing Stress (q'_o) due to LRFD Service I load combination is used to evaluate footing settlement.

Strength Limit State Bearing Capacity Check		
q_o (ksf)	<	q_R (ksf)
5.74		16.83
OKAY		

Extreme Limit State Bearing Capacity Check		
q_o (ksf)	<	q_R (ksf)
2.35		16.83
OKAY		

References

1) Caltrans, Memo To Designers 4-1 Spread Footings, April 2008.
2) Nominal Bearing Resistance Equation (10.6.3.1.2a-1)
AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.
3) Schmertmann's Modified Method for Calculation of Immediate Settlements (1978),
Soils and Foundations - Volume II, FHWA NHI-06-089, December 2006.
4) Elastic Constants of Various Soils (Table C10.4.6.3-1)
AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

NOMINAL BEARING RESISTANCE -- STRENGTH LIMIT STATE (AASHTO Bridge Design Specifications)

Date: 3/19/2012 *Support:* BENT 2
Project: SILVA VALLEY WB ON-RAMP UC *Boring:* R-10-005
BCI No: 556.2

Equation: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\, \gamma B N_{\gamma m} C_{w\gamma}$

in which:

$N_{cm} = N_c s_c i_c$

$N_{qm} = N_q s_q d_q i_q$

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$

D_w	C_{wq}	$C_{w\gamma}$
0	0.5	0.5
D_f	1.0	0.5
>1.5B+D_f	1.0	1.0

where:

q_n = nominal bearing resistance
c = cohesion (psf)
B' = effective footing width (feet)
γ = total (moist) unit weight of soil (pcf)
D_f = footing embedment depth (feet)
N_c, N_q, and N_γ = bearing capacity factors
C_{wq} & $C_{w\gamma}$ = correction factors for location of ground water
s_c, s_γ, and s_q = footing shape correction factors
d_q = correction factor to account for shearing resistance in material above bearing level
i_c, i_γ, and i_q = load inclination factors
D_w = depth to ground water taken from the ground surface (feet)

Input Parameters

γ =	130	(pcf)
ϕ =	38	(degrees)
c =	0	(psf)
D_f =	6.5	(feet)
D_w =	0	(feet)

d_q =	1.0
i_c =	1.0
i_γ =	1.0
i_q =	1.0

Bottom Footing Elevation (feet):	677.5
Finished Grade (feet):	684.0
Ground Water Elevation (feet):	684.0

Resistance Factor (φ_b)= 0.45

Strength Limit State

Solve for Gross Nominal Bearing Resistance

Effective Footing Dimensions B'	Effective Footing Dimensions L'	C_{wq}	$C_{w\gamma}$	s_c	s_γ	s_q	Gross Nominal Bearing Resistance			Factored Gross Nominal Bearing Resistance, Resistance Factor (φ_b) = 0.45		
(feet)							(psf)	(ksf)	(tsf)	(psf)	(ksf)	(tsf)
6.6	38.0	0.50	0.50	1.00	1.00	1.00	37410	37.41	18.7	16835	16.83	8.4

Bearing Capacity Factors

N_c = 61.35
N_q = 48.93
N_γ = 78.03

Shape Correction Factors

ϕ	s_c	s_γ	s_q
$\phi = 0$	1 + (B/5L)	1.0	1.0
$\phi > 0$	1 + (B/L)(N_q/N_c)	1 - 0.4(B/L)	1 + (B/L)tanϕ

Note: If L > 5B, then s_c, s_γ, and s_q = 1.0 (Geotechnical Engineering Circular No. 6, FHWA-SA-02-054, pgs 55-56)

Nominal Bearing Resistance and Immediate Settlement Worksheet (LRFD)

Date:	3/19/2012	*Support:*	BENT 3
Project:	SILVA VALLEY WB ON-RAMP UC	*Boring:*	R-10-005
BCI No:	556.2	*By:*	WEN
		Check by:	
		Date:	

		Eff. Footing Width, B'f (feet)	Eff. Footing Length, L'f (feet)
LRFD Service Limit State I Vertical Load (kips):	1060	7.1	37.4
LRFD Strength Limit State Load (kips):	1460	5.9	35.7
LRFD Extreme Event Limit State Load (kips):	820	8.4	41.9

Ground Surface Elevation (feet):	684.0	(equal to footing bottom for a footing in fill above ex. grnd. surface)
Ground Water Elevation (feet):	684.0	
Depth to Ground Water (feet):	0.0	(for settlement analysis)
Depth of footing (feet):	6.5	
Time to Settlement (t):	1.2	
Bottom Footing Elevation (feet):	677.5	
Finished Grade (feet):	684.0	
Depth to Ground Water (feet):	0.0	(for bearing resistance analysis)
Depth of footing (feet):	6.5	
γ (pcf) =	130	Soil Parameters at base of footing
ϕ (degrees) =	38	
c (psf) =	0	
Resistance Factor (φ_b)=	0.45	

Layer	Soil Description	Depth Bottom Layer (feet)	Layer Thickness (feet)	Top Elev. (feet)	Bottom Elev. (feet)	Soil Unit Weight (pcf)	Soil Type (1, 2, 3, or 4)	$N1_{60}$	Es (tsf)	or	Estimated Es (tsf)
1	Int. Wthd Rock	5.5	5.5	684.0	678.5	125	3	70	700		
2	Wthd Rock	100	94.5	678.5	584.0	130	4				2000
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Soil Types

1) Silts, sandy silts, slightly cohesive mixtures
2) Clean fine to medium sands and slightly silty sands
3) Coarse sands and sands with little gravel
4) Sandy gravel and gravels

Gross Nominal Bearing Resistance q_n (ksf)	Factored Gross Nominal Bearing Resistance q_R (ksf)
35.64	16.04

Gross Uniform Bearing Stress (Service Limit) q_o (ksf)	Net Bearing Stress (Service Limit) q'o (ksf)	Gross Uniform Bearing Stress (Strength Limit) q_o (ksf)	Gross Uniform Bearing Stress (Extreme Limit) q_o (ksf)
3.99	3.58	6.93	2.33

Sevice Limit State Settlement (1 inch) Check		
q'_o (ksf)	<	q_{pn} (ksf)
3.58		26.60
OKAY		

Permissible Net Contact Stress q_{pn} (ksf)	Permissible Gross Contact Stress q_{pg} (ksf)	Immediate Settlement S_i (inches)
26.60	27.01	1.00

Immediate Settlement under Net Bearing Stress due to Service Limit I State Load Combination S_i, (inches)
0.09

The Net Bearing Stress (q'_o) due to LRFD Service I load combination is used to evaluate footing settlement.

Strength Limit State Bearing Capacity Check		
q_o (ksf)	<	q_R (ksf)
6.93		16.04
OKAY		

Extreme Limit State Bearing Capacity Check		
q_o (ksf)	<	q_R (ksf)
2.33		16.04
OKAY		

References

1) Caltrans, Memo To Designers 4-1 Spread Footings, April 2008.
2) Nominal Bearing Resistance Equation (10.6.3.1.2a-1)
AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.
3) Schmertmann's Modified Method for Calculation of Immediate Settlements (1978),
Soils and Foundations - Volume II, FHWA NHI-06-089, December 2006.
4) Elastic Constants of Various Soils (Table C10.4.6.3-1)
AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007.

NOMINAL BEARING RESISTANCE -- STRENGTH LIMIT STATE (AASHTO Bridge Design Specifications)

Date: 3/19/2012
Project: SILVA VALLEY WB ON-RAMP UC
BCI No: 556.2
Support: BENT 3
Boring: R-10-005

Equation: $q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\,\gamma BN_{\gamma m} C_{w\gamma}$

in which:

$N_{cm} = N_c s_c i_c$

$N_{qm} = N_q s_q d_q i_q$

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$

D_w	C_{wq}	$C_{w\gamma}$
0	0.5	0.5
D_f	1.0	0.5
>1.5B+D_f	1.0	1.0

where:

q_n = nominal bearing resistance
c = cohesion (psf)
B' = effective footing width (feet)
γ = total (moist) unit weight of soil (pcf)
D_f = footing embedment depth (feet)
N_c, N_q, and N_γ = bearing capacity factors
C_{wq} & $C_{w\gamma}$ = correction factors for location of ground water
s_c, s_γ, and s_q = footing shape correction factors
d_q = correction factor to account for shearing resistance in material above bearing level
i_c, i_γ, and i_q = load inclination factors
D_w = depth to ground water taken from the ground surface (feet)

Input Parameters

γ =	130	(pcf)
ϕ =	38	(degrees)
c =	0	(psf)
D_f =	6.5	(feet)
D_w =	0	(feet)

d_q =	1.0
i_c =	1.0
i_γ =	1.0
i_q =	1.0

Bottom Footing Elevation (feet):	677.5
Finished Grade (feet):	684.0
Ground Water Elevation (feet):	684.0

Resistance Factor (φ_b)= 0.45

Strength Limit State

Solve for Gross Nominal Bearing Resistance

Effective Footing Dimensions B'	Effective Footing Dimensions L'	C_{wq}	$C_{w\gamma}$	s_c	s_γ	s_q	Gross Nominal Bearing Resistance (psf)	Gross Nominal Bearing Resistance (ksf)	Gross Nominal Bearing Resistance (tsf)	Factored Gross Nominal Bearing Resistance, Resistance Factor (φ_b) = 0.45 (psf)	(ksf)	(tsf)
(feet)												
5.9	35.7	0.50	0.50	1.00	1.00	1.00	35635	35.64	17.8	16036	16.04	8.0

Bearing Capacity Factors

N_c = 61.35
N_q = 48.93
N_γ = 78.03

Shape Correction Factors

ϕ	s_c	s_γ	s_q
$\phi = 0$	1 + (B/5L)	1.0	1.0
$\phi > 0$	1 + (B/L)(N_q/N_c)	1 - 0.4(B/L)	1 + (B/L)tanϕ

Note: If L > 5B, then s_c, s_γ, and s_q = 1.0 (Geotechnical Engineering Circular No. 6, FHWA-SA-02-054, pgs 55-56)

Design Calculations

Elastic Constants of Various Soils

ELASTIC CONSTANTS OF VARIOUS SOILS
(after AASHTO LRFD BDS)

Typical Range of Values

Soil Type	Young's Modulus, Es (tsf)	Poisson's Ratio, ν (dim)
Clay:		
Soft sensitive	25-150	
Medium stiff to stiff	150-500	0.4-0.5 (undrained)
Very stiff	500-1000	
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine sand:		
Loose	80-120	
Medium dense	120-200	0.25
Dense	200-300	
Sand:		
Loose	100-300	0.2-0.35
Medium dense	300-500	
Dense	500-800	0.3-0.4
Gravel:		
Loose	300-800	0.2-0.35
Medium dense	800-1000	
Dense	1000-2000	0.3-0.4

Estimating Es from SPT N-value ($N1_{60}$)(1)

Soil Type	Young's Modulus, Es (tsf)
1) Silts, sandy silts, slightly cohesive mixtures	$4N1_{60}$
2) Clean fine to medium sands and slightly silty sands	$7N1_{60}$
3) Coarse sands and sands with little gravel	$10N1_{60}$
4) Sandy gravel and gravels	$12N1_{60}$

ELASTIC CONSTANTS OF VARIOUS SOILS
MODIFIED AFTER US DEPARTMENT OF THE NAVY (1982) AND BOWLES (1982)

Estimating Es from S_u(2)

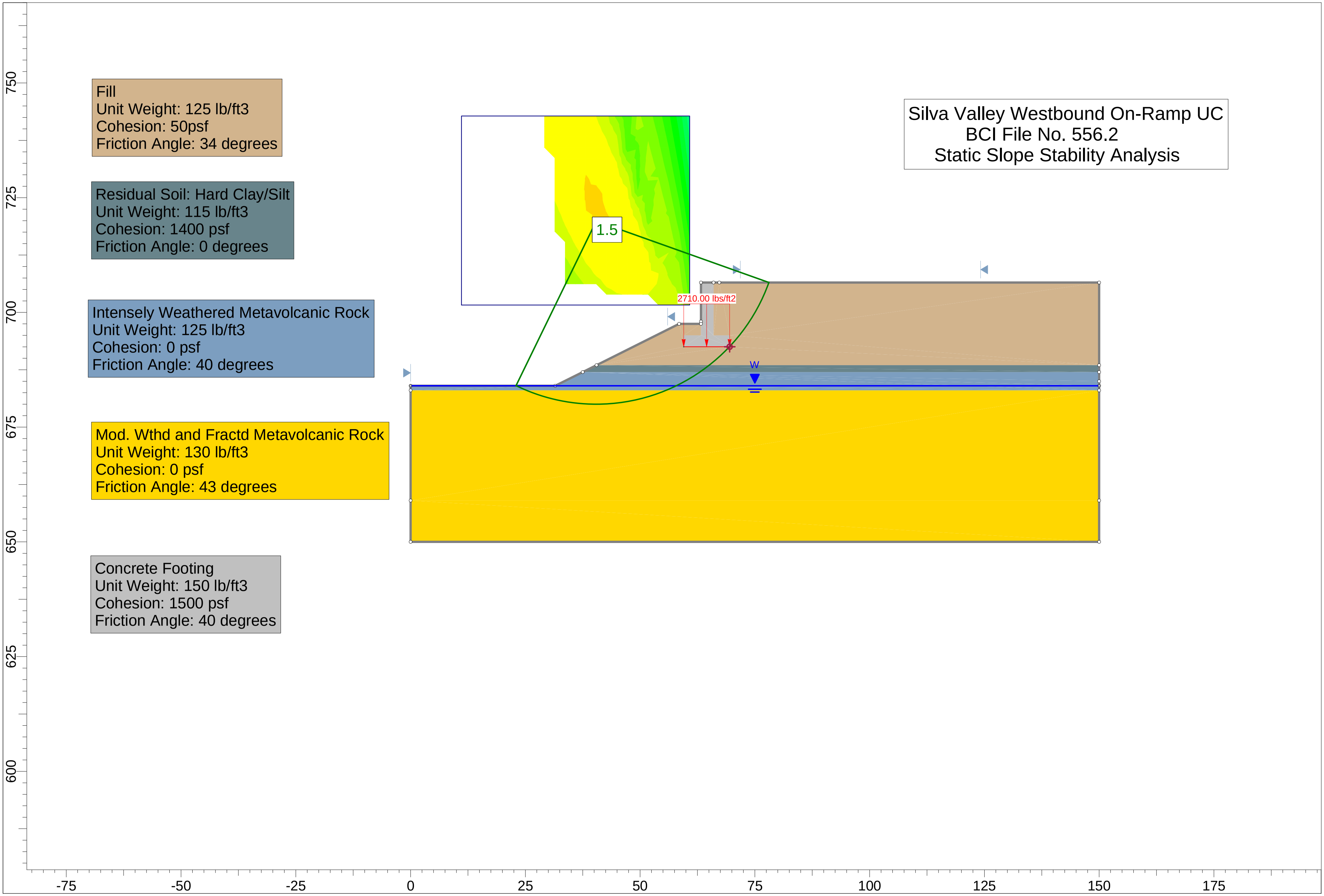
Soil Type	Young's Modulus, Es (tsf)
Soft sensitive clay	$200S_u$-$500S_u$
Medium stiff to stiff clay	$750S_u$-$1{,}200S_u$
Very stiff clay	$1{,}500S_u$-$2{,}000S_u$

(1) $N1_{60}$ = SPT corrected for depth and overburden.
(2) S_u = Undrained shear strength (tsf).

Sources: Typical Ranges of Values / Estimating Es from SPT N-value
Table C10.4.6.3-1, AASHTO LRFD Bridge Design Specifications, 4th Edition.
Table 5-16, FHWA NHI-06-088, Soils and Foundations - Volume 1, December 2006.
Estimating Es from S_u
Caltrans Bridge Design Specifications, November 2003.

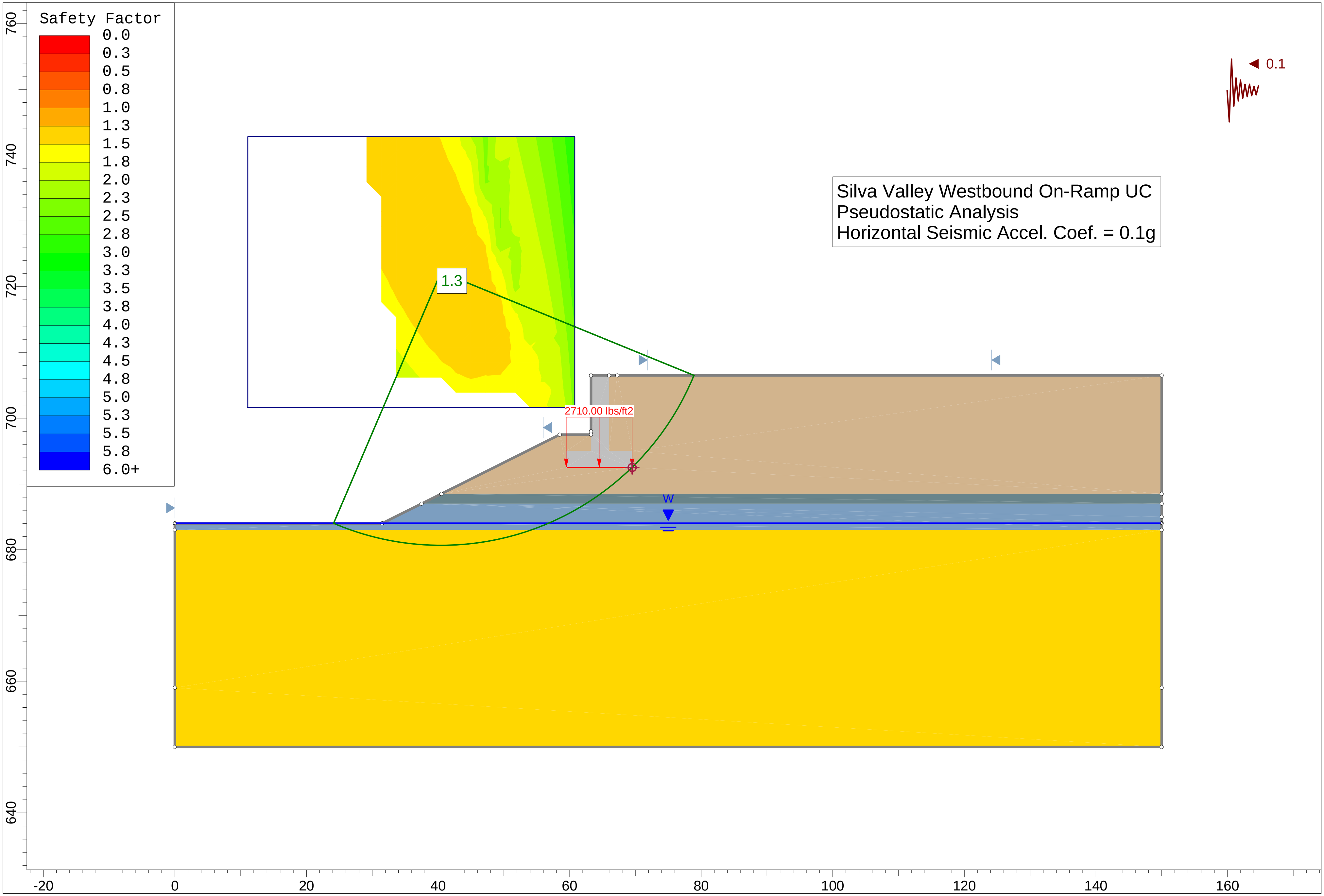
Design Calculations

Slope Stability Output Graphs



Static WB with GW.slim

11/4/2010, 9:43:27 AM



SVP WB Pseudo.slim

11/4/2010, 9:43:27 AM

Design Calculations

Lateral Earth Pressure

EQUIVALENT FLUID WEIGHTS (EFWs)

Project: Silva Valley EB On-Ramp UC / WB Off-Ramp UC
BCI No.: 556.2
Date: 2/27/2012
By: WEN

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 =	0.21	
Horizontal seismic acceleration coefficient,	k_h =	0.11	
Vertical seismic acceleration coefficient,	k_v =	0.00	
Lateral wall displacement (inches),	d =	1.00	($1 \le d \le 8$)

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$_{EQ}$	4	--	--	psf/f
*	**Passive$_{EQ}$**	383	255	191	psf/f
	At rest$_{EQ}$	7	--	--	psf/f

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

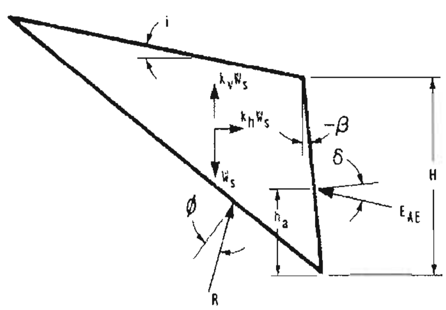
K_A =	0.29
K_P =	3.39
K_O =	0.46
K_{AE} =	0.33
K_{PE} =	3.19

Note: Active$_{EQ}$ and At rest$_{EQ}$ EFWs are additional to static Active and At rest EFWs.

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 angle

* Level Ground Surface Only.

APPENDIX E

Draft Report Comment and Response –

Caltrans OGDN and OSFP

OGDN Review Comment & Response Form

General Project Information	Review Phase		Reviewer Information
Dist: 03 **EA:** 1E2901 **EFIS Project No:** 0300000258 **Project Name:** Silva Valley Pkwy Interchange **Liaison Engineer:** Erick Fredrickson	☐ **PSR/PDS (Review No. _)** ☐ **APS/PSR (Review No. _)** ☐ **APS/PR (Review No. _)** ☐ **Type Selection**	☒ **65% PS&E Unchecked Details** ☐ **PS&E (Review No. _)** ☐ **Construction Support** ☐ **Other:**	**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

Structure Information

Structure Name	Bridge No.
Silva Valley Pkwy OC	25-0127
EB Off-Ramp UC	25-0128S
WB On-Ramp UC	25-0129K
WB Off-Ramp Br	25-0130K
WB Off-Ramp Retaining Wall	
Carson Creek MSE Wall	
Bucks Ravine Creek RCB	

Consultant Information (to be filled in by Consultant)

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

	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	NA	

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.	**NA**	
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.	**NA**	
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.	**NA**	
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.	**NA**	
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.	**NA**	

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RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(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.	**NA**	
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.	BCI correlated Es with N160 value and/or soil type consistent with Table 5-16, FHWA NHI-06-088, Soils and Foundations, Volume 1, December 2006. BCI modeled the new embankment fill with N160 = 16 to correlate to Es value in our settlement calculations.	
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?	**BCI has corrected the typo.**	
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.	**BCI recommends that both abutment foundations be established within a prsim of engineered fill. See report for final foundation recommendations.**	
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.	**BCI used modified bearing capacity factors in consideration of the 2:1 endslope. BCI includes updated calculations for spread footings at all support locations in App. D.**	
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.	**BCI has revised the language in the report.** **BCI defers to the structual engineer to define the types of lateral load(s).**	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
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RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer's use)

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.	**For final design, BCI modeled the fill materials with a friction angle of 34 degrees and a nominal cohesion of 50 psf.** **BCI also updated strength parameters for the underlying rock (phi ranges from 40 to 43 degrees).**	
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.	**BCI modeled ground water at elev. 684 ft.**	
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?	**NA**	
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.	**NA**	
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.	**NA**	
16	Carson Creek MSE Wall General Plan, Sheet 1 of 8, <u>TYPICAL SECTION</u>	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."	**NA**	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
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RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(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.	**NA**	

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)

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"	NA	
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.	NA	
3		Pg 7	9.2 – Provide 'Mmax' used for ARS curve.	NA	
4		Pg 9	10, bullets – Verify / update all bridge information w/ final plans.	NA	
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?	NA	

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)

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 "03-1E2901"	BCI has revised the "EA."	
7		Pg 1	2.1 – Revise the description from "100' south" to "xx' north". Be clear between 'old / existing' and 'new' Silva Valley Parkway.	BCI has revised the text for clarity.	
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?).	BCI has edited the text and verified the 4:1 side slope on the north side of Abutment 4.	
9		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	BCI provides Mmax.	
10		Pg 10	12 – Revise "EB Off-Ramp" with "WB On-Ramp"	BCI has revised the text.	
11	**FR #25-0128S**	Cover Pg	Include PM. Revise "EA" to "03-1E2901"	NA	
12		Pg 1	2.1 – Be clear between 'old / existing' and 'new' Silva Valley Parkway.	NA	
13		Pg 2	1st para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2nd para – Delete 1st & 2nd sentence.	NA	
14		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	NA	
15		Pg 10, 11	Table 3, 4, 5 – Revise / update footing 'L' dimension.	NA	
16	**FR #25-0130K**	Cover Pg	Revise "EA" to "03-1E2901"	NA	
17		Pg 2	2.2 – Revise bridge width dimension. 3 – Complete the description of the borings ("two…"borings?). Are there also "two" test pits?	NA	
18		Pg 6	9.2 – Provide 'Mmax' used for ARS curve.	NA	
19		Pg 8, 9	10 – Revise / update abutment and bent footing dimensions.	NA	
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.	NA	
22					
23					
24					

Submittal Data (Reviewer to complete)

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

Br No*.____ *=if applicable