FOUNDATION REPORT
US 50, Silva Valley Parkway Interchange
Westbound Off-Ramp Bridge
El Dorado County, California
Bridge No. 25-0130K
03-ED-50
PM R1.90
EA 03-1E2901

Prepared for:

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File No. 556.2
April 30, 2012

Mrs. Julie Passalacqua
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Subject: **FOUNDATION REPORT**
US 50, Silva Valley Parkway Interchange
Westbound Off-Ramp Bridge, Bridge No. 25-0130K
03-ED-50; PM R1.90; EA 03-1E2901

Dear Mrs. Passalacqua,

In accordance with our April 7, 2010 agreement, Blackburn Consulting (BCI) prepared this Foundation Report for the westbound off-ramp bridge 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) on August 26, 2010 and our draft Foundation Report in November 2010.

Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Foundation Report for the westbound off-ramp bridge planned for the US 50/Silva Valley Parkway Interchange project in El Dorado Hills, El Dorado County, California.

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

1.2 Scope of Services

To prepare this report, BCI:

- Reviewed preliminary bridge design plans provided by Mark Thomas and Company, Inc. (MTCo)
- Discussed the project design needs with MTCo
- Reviewed geologic and seismic maps pertaining to the site
- Drilled and sampled two (2) auger/diamond core borings to a maximum depth of 32 feet below existing grade at the abutments, and excavated two (2) test pits to a maximum depth of 4 feet at the bents
- Performed laboratory testing on soil and rock samples retrieved from the borings
- Performed engineering and seismic analysis to provide recommendations for structure foundations and approach

This Foundation Report supersedes the Preliminary Foundation Report by BCI for the Silva Valley Parkway Interchange, Carson Creek Bridges, Westbound Off- and Eastbound On-Ramps dated August 26, 2010 and the Draft Foundation Report for this bridge dated November 5, 2010.

2 PROJECT DESCRIPTION

2.1 Project Location and Site Description

The project is located in El Dorado County, California, along US 50, near Post Mile R1.90, approximately 1,200 east of the existing Clarksville Undercrossing (UC, Br. No. 25-0072, at the existing Silva Valley Parkway) at Carson Creek. Figure 1 (Vicinity Map), in Appendix A, shows the approximate project location.

The bridge location consists of natural slopes with no improvements. Rock outcrop is exposed intermittently on the slopes and the creek bed exposes hard metavolcanic rock. Creek flow is to the south and was minor (6 to 12 inches deep) at the time of our field review (July 2010).

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

2.2 Proposed Structure

The project will consist of a new ramp bridge that spans Carson Creek on the north side of US 50. The bridge will be a three span, reinforced concrete box girder bridge approximately 257 feet long by 51 feet wide. Roadway elevation at the west abutment (Abutment 1) will be approximately 8 feet above existing grade, and at the east abutment (Abutment 4), approximately 14 feet above existing grade and will require approach/abutment fill.

The General Plan and Foundation Plan prepared by MTCo (see Appendix B) shows the overcrossing structure with two-column piers at each interior support. MTCo proposes 2H:1V slopes at the approach sides and in front of each abutment with the exception of the south side slope at Abutment 4 which has a 4H:1V slope. Foundations will consist of 8.25-foot wide spread footings at Abutments 1 and 4, and two, 11- foot square spread footings at Piers 2 and 3.

2.3 Existing Facilities

There are no existing facilities at the bridge location.

3 SUBSURFACE INVESTIGATION

To characterize the subsurface conditions and obtain samples for laboratory testing, BCI retained PC Exploration to drill and sample two borings and Monte Ricky to excavate two test pits at the site in July 2010. PC Exploration used a CME 75 truck-mounted rig to drill the borings with 8-inch O.D. hollow-stem augers to relatively competent bedrock, and then HQ, wireline, diamond core equipment to complete the borings in rock. Core diameter is approximately 3.8 inches. Monty Rickey Excavating excavated the test pits using a Caterpillar 430D and a 24-inch wide bucket. The maximum depth of the borings and test pits is ±51.0 feet and ±4 feet below the ground surface (bgs), respectively.

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

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

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

4 LABORATORY TESTING

We completed laboratory tests on representative samples obtained from the exploratory boring. In addition to field blow counts (in the upper 6 feet) and Rock Quality Designation (RQD) values, we completed unconfined compression tests of rock (ASTM D 2938) for strength parameters. We attach laboratory test results in Appendix C.

5 SITE GEOLOGY

5.1 Topography

Within the bridge area, natural slopes are on the order of 35 to 40 feet high (creek bottom to proposed abutment location) and at gradients ranging from 2H:1V (horizontal to vertical) to 5H:1V. Moderately dense grasses and scattered trees/bushes cover the slopes. The creek flows to the south at an approximate gradient of 1.5%. Foundation locations are 20 to 30 feet above the creek bed at the abutments and 5 to 10 feet at the piers.

5.2 Regional Geology

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

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

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.

5.3 Site Geology

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

Rock structure at the bridge location is similar to the surrounding area and has a predominant foliation with a strike of north, 45º west, and a steep dip of 70º-85º to the north; along which most fractures occur. Other fractures/discontinuities exposed in our trenches are mostly random and discontinuous. Fractures appear generally closed (tight) and rough.

We did not observe indications of slope instability on the natural slopes in the area. We did not observe groundwater seepage in the bridge area (outside of the creek bed).

The West Bear Mountains Fault is located about 4,500 feet west of the site (near Latrobe Road) with a short splay mapped to the east approximately 2,600 feet west of the site. The East Bear Mountains Fault (or Rescue section) is located approximately 7 miles east of the site. Faults are not mapped through or adjacent to the OC site and we observed no indication of active faulting in the area.

We did not observe significant occurrence of ultramafic rock where naturally occurring asbestos minerals (NOA) are likely to occur. Published mapping and site review does not indicate that the project is within an ultramafic rock area; however, ultramafic rock and faulting are mapped nearby and naturally occurring asbestos minerals could potentially occur in the area. Geologic mapping of asbestos containing rocks 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.

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

6 SUBSURFACE CONDITIONS

6.1 Subsurface Soil and Rock Conditions

In general, hard rock occurs at relatively shallow depths. Rock outcrop is exposed intermittently on the slopes and the creek bed is cut down to hard rock.

At the abutments, our borings encountered 1 to 1.5 feet of silty sand over decomposed to intensely weathered rock that becomes less weathered (moderate to slight) at a depth of approximately 15 feet. Core recovery was generally greater than 60% to a depth of 15 feet and 90 to 100 percent below depths of 15 feet. Rock Quality Designation (RQD) indicates poor to fair quality rock (RQD of 0 to 50%) to depths of 15 to 20 feet and fair to excellent quality rock (RQD of 50% to 100%) below those depths.

Our test pits at the pier locations encountered 1 to 1.5 feet of sandy silt with gravel and cobbles/boulders with silt underlain by intensely to moderately weathered and intensely fractured rock. At a depth of 3 feet, we encountered hard, moderately weathered and fractured rock.

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

6.2 Groundwater

We did not encounter free groundwater to elevation 723 feet within the augered portions of the borings or in our test pits completed in July 2010. We did not evaluate groundwater or perched water conditions in the diamond-cored portion of the borings due to the presence of drilling fluid.

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

7 SCOUR EVALUATION

Hard rock underlies the creek bed and adjacent slopes and the rock will limit the depth of potential scour. At our test pits, located on the lower slopes near the pier locations, we encountered hard rock at depths of 3 feet. Hard rock is also exposed at the surface along most of the creek bed. Potential scour depths are not likely to exceed 3 feet at the pier locations and the hard rock that the piers will be founded within is not considered susceptible to scour. The base of abutment foundations are well above (10 to 19 feet) the 100-year storm water level of elevation 710 feet (per MTCo).

8 CORROSION EVALUATION

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

9 SEISMIC RECOMMENDATIONS

9.1 Fault Rupture

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

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

9.2 Ground Motion

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

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

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

The probabilistic spectrum is obtained from the USGS (2008) National Hazard Map for 5% probability of exceedance in 50 years. Probabilistic analysis includes deaggregation for applicable fault distance when near-fault effects apply (as for this site).

Both the deterministic and probabilistic spectra account for soil effects through incorporation of the parameter Vs30, the average shear wave velocity in the upper 30 meters of the soil profile. For the project site, we assume a Site Class B/C with Vs30 equal to 760 meters per second (approximately 2,500 feet per second) based on the mapped ground conditions (underlain by shallow metamorphic rock).

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

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

Table 2 - Caltrans ARS Online Envelope* Spectrum Data

Period	SA	Period	SA	Period	SA	Period	SA
0	0.197	**0.085**	0.376	**0.35**	0.333	**1.4**	0.091
0.01	0.197	**0.09**	0.389	**0.36**	0.327	**1.5**	0.086
0.02	0.201	**0.095**	0.401	**0.38**	0.315	**1.6**	0.082
0.022	0.204	**0.1**	0.414	**0.4**	0.303	**1.7**	0.078
0.025	0.208	**0.11**	0.43	**0.42**	0.291	**1.8**	0.074
0.029	0.214	**0.12**	0.445	**0.44**	0.279	**1.9**	0.071
0.03	0.216	**0.13**	0.458	**0.45**	0.273	**2**	0.068
0.032	0.221	**0.133**	0.461	**0.46**	0.267	**2.2**	0.061
0.035	0.228	**0.14**	0.468	**0.48**	0.257	**2.4**	0.055
0.036	0.231	**0.15**	0.476	**0.5**	0.248	**2.5**	0.052
0.04	0.241	**0.16**	0.476	**0.55**	0.223	**2.6**	0.05
0.042	0.246	**0.17**	0.474	**0.6**	0.203	**2.8**	0.045
0.044	0.251	**0.18**	0.472	**0.65**	0.185	**3**	0.042
0.045	0.254	**0.19**	0.469	**0.667**	0.18	**3.2**	0.038
0.046	0.256	**0.2**	0.466	**0.7**	0.171	**3.4**	0.035
0.048	0.262	**0.22**	0.444	**0.75**	0.158	**3.5**	0.034
0.05	0.267	**0.24**	0.423	**0.8**	0.148	**3.6**	0.033
0.055	0.284	**0.25**	0.413	**0.85**	0.14	**3.8**	0.03
0.06	0.3	**0.26**	0.403	**0.9**	0.134	**4**	0.029
0.065	0.317	**0.28**	0.386	**0.95**	0.129	**4.2**	0.028
0.067	0.323	**0.29**	0.377	**1**	0.124	**4.4**	0.026
0.07	0.333	**0.3**	0.369	**1.1**	0.112	**4.6**	0.025
0.075	0.348	**0.32**	0.354	**1.2**	0.104	**4.8**	0.024
0.08	0.362	**0.34**	0.34	**1.3**	0.097	**5**	0.023

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

9.3 Liquefaction Evaluation

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

9.4 Seismic Settlement

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

9.5 Seismic Slope Instability

Due to the presence of shallow rock, favorable rock structure, and relatively shallow slope gradients (2H:1V to 6H:1V), we consider the potential for seismic slope instability in the form of landslides and mudslides at this site to be very low.

10 FOUNDATION RECOMMENDATIONS

The most appropriate foundation type for the bridge structure appears to be shallow spread footings established within the weathered rock at least 5 feet below existing grade at the abutments and 6 feet at the bents.

Cast-in-Drilled Hole (CIDH) pile foundations or large diameter drilled-shafts were considered; however, difficult drilling is expected due to both the hardness of the rock and the frequency of fractures. Driven 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.

The following summarizes the proposed foundation design, as developed by MTCo and shown on the General Plan and Foundation Plan in Appendix B:

- The foundation for Abutments 1 and 4 consist of spread footings. The footings are 8.25 feet wide, 53 feet long, and 2.5 feet thick. The design requires a contact pressure of approximately 6 kips per square foot (ksf).
- At Pier 2 and 3, MTCo proposes two columns, each with an individual spread footing. The footings are 11 feet square, 3 feet thick, with edges 12 feet apart (23 feet on-center). The design requires a contact pressure of approximately 19 kips per square foot (ksf).
- The proposed bottom of footing elevation at Abutment 1 is 730.0 feet; Abutment 4 is 722.0 feet. The bottom of Pier 2 footings is elevation 705.0 feet; Pier 3 is 706.0 feet.

10.1 Shallow Foundations

10.1.1 Spread Footing Data Table

Based on footing foundation design data provided by MTCo and our geotechnical analysis, we provide foundation design recommendations in Table 3. We include spread footing design calculations in Appendix D. A discussion of our analyses follows.

Table 3 – 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	8.3	53	730.0	4	10	10	N/A	N/A	N/A
Pier 2	11	11	705.0	4	N/A	N/A	20	22.5	50
Pier 3	11	11	706.0	4	N/A	N/A	20	22.5	50
Abut 4	8.3	53	722.0	4	10	10	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.

10.1.2 Slope Stability

The abutments will be founded on slopes with a gentle to moderate gradient (2H:1V to 6H:1V) underlain by shallow rock. Based on the presence of hard, shallow rock, favorable rock structure (pervasive discontinuities dip at angles generally greater than 70º), and relatively shallow slope gradients, we expect abutments to be grossly stable at the planned foundation configuration.

10.1.3 Lateral Resistance

Calculate lateral load resistance of spread footings as follows:

- A soil friction factor (tan δ) of 0.45 for cast in-place concrete foundations bearing on intact rock materials or compacted structure backfill. Use a resistance factor (φ_τ) of 0.8 for LRFD. Foundations will be on weathered rock; use an assumed angle of internal friction equal to 35 degrees for further analysis of frictional resistance.

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

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

10.1.4 Settlement

Based on the proposed design loads and the underlying rock conditions, total settlement at abutment and bent foundations will not exceed ½-inch. We do not expect differential settlement between adjacent footings to exceed ½-inch. Settlement of spread footing foundations at the piers is based on empirical values for footings on competent rock (Caltrans BDS 4.4.8.2). At the abutments, empirical values for competent rock and evaluation of settlement for broken or jointed rock (per Caltrans BDS 4.4.8.2.2) indicate settlement will be less than ½-inch.

10.2 Approach Fill Earthwork

10.2.1 Fill Material

We assume locally excavated soil/weathered rock will be used for construction of approach fills at this location. 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.

10.2.2 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. Place only material with a low expansion potential. Low expansion material is defined as having an Expansion Index (EI) less than 50 (per ASTM D4829), and a Sand Equivalent (SE) greater than 20 (per California Test 217).

10.2.3 Geometry and Stability

Where approach fill is placed, side and front slopes will have a gradient of 2H:1V or flatter. The proposed geometry is a common slope gradient considered stable for typical approach fill construction. We assume abutment backfill will consist of materials conforming to Structure Backfill requirements. The mostly moderate slope of the existing ground surface and high strength of the underlying rock will provide a stable base on which to construct the fills. Foundations supported on or near a fill slope are not proposed.

10.2.4 Site Preparation

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

We observed non-structural fill materials in the approach area east of Abutment 4 (see the project GDR). Remove all unsuitable (non-structural) fill materials prior to approach fill construction in accordance with the "Standard Specifications", Section 19-2.02, and the GDR. The project geotechnical engineer must approve the prepared ground surface prior to placement of approach fill.

10.2.5 Settlement

Due to the presence of shallow rock, we do not anticipate significant settlement at approach fills. 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.

10.2.6 Lateral Earth Pressures

We assume that the approach fill material meets the requirements of Caltrans standard for Structure Backfill. Use the following equivalent fluid weights (EFW) to design the abutments walls and wing walls at Abutments 1 and 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 wall drainage is placed in accordance with Caltrans “Standard Plans and Specifications.”

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

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

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

For surcharge loads, apply an additional uniform lateral load behind the wall equivalent to 0.3-times the surcharge pressure. Use a soil friction factor (tan δ) of 0.45 for cast in-place concrete foundations bearing on weathered rock or compacted fill materials. Foundations will be on weathered rock; use an assumed angle of internal friction equal to 35 degrees for further analysis of frictional resistance.

11 CONSTRUCTION CONSIDERATIONS

11.1 Cuts and Excavations

Typical grading equipment such as scrapers, dozers, backhoes and excavators are sufficient to excavate surficial soil and decomposed to intensely weathered rock at the proposed bridge site. However, due to the presence of moderately hard to hard rock (particularly at the pier 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 pier foundation locations and isolated locations within the abutment foundation excavations.

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

At support locations, rock blasting may disrupt/degrade integrity of the surrounding rock. Therefore, rock blasting should not be permitted to construct new 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.

11.2 Embankments

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

11.3 Spread Footings

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

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

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

11.4 Dewatering

We do not anticipate the presence of significant ground water within footing excavations during dry season construction (June through October). Some seepage is possible at the pier footing excavations since they will be at or below creek level. If 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.

11.5 Naturally Occurring Asbestos

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

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

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

11.6 Storm Water Quality

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

12 RISK MANAGEMENT

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

- Review and provide written comments on the (civil, structural) plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, we should 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.

13 LIMITATIONS

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

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

Use this foundation report only for the design and construction of the US 50 / Silva Valley Parkway Interchange, Westbound Off-Ramp Bridge.

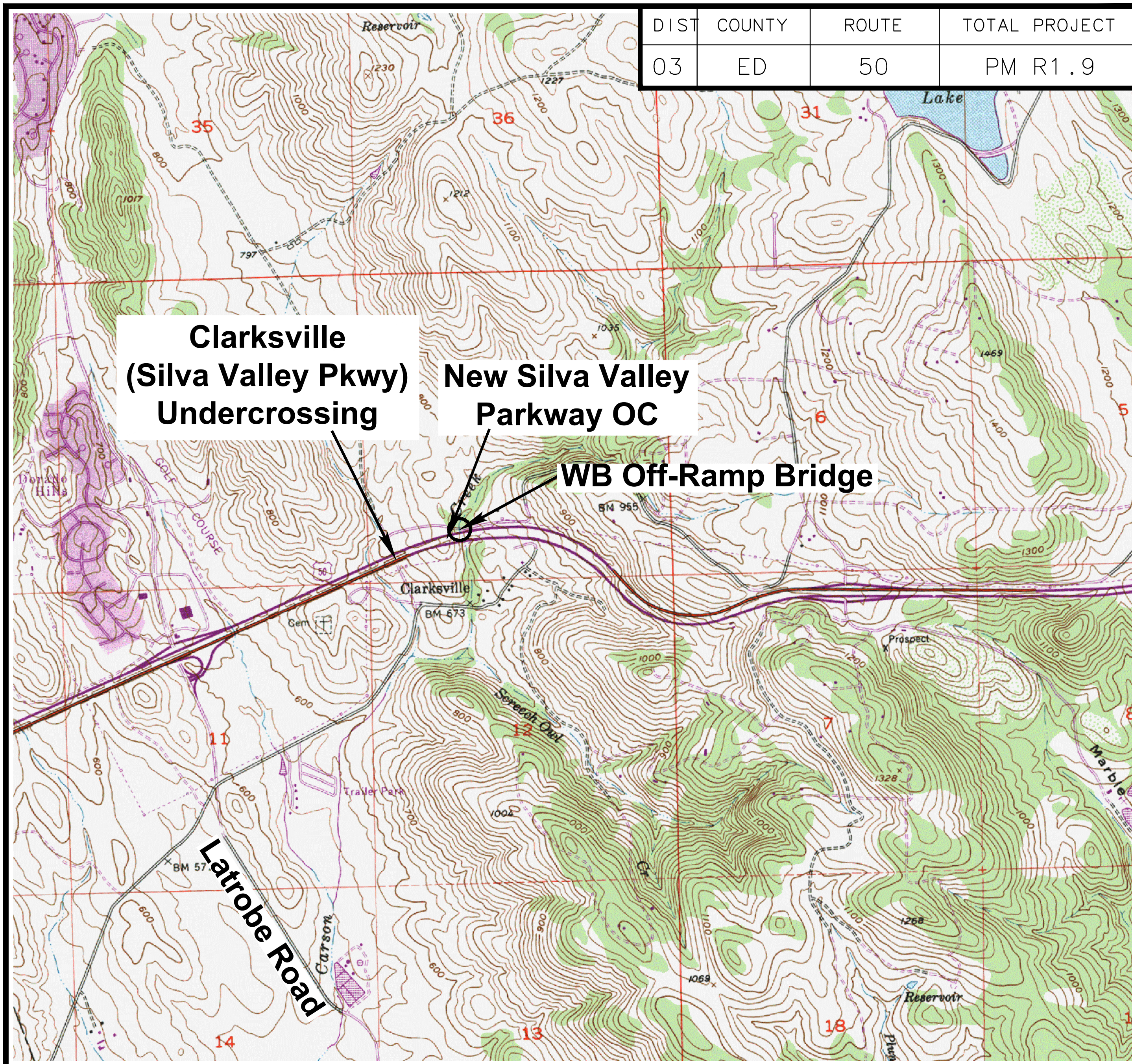
Modern design and construction is complex, with many regulatory sources, restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and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

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

APPENDIX A

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

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.9



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

N

SCALE: 1"=0.5 Miles

blackburn consulting

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Fax: (530) 887-1495
www.blackburnconsulting.com

VICINITY MAP

Silva Valley Parkway Interchange
WB Off-Ramp Bridge, EA 03-1E2901
El Dorado County, California

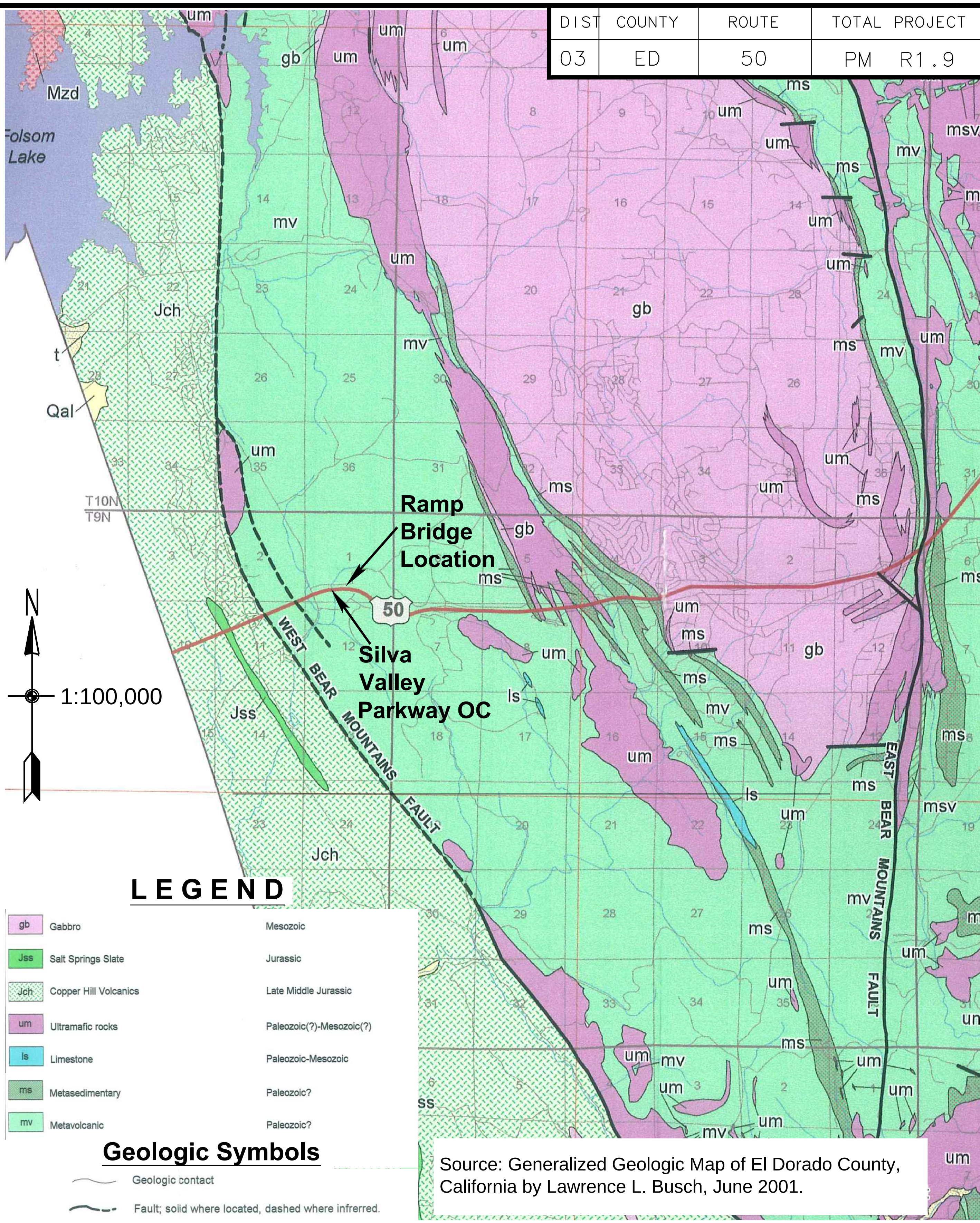
File No. 556.2

April 2012

Figure 1

5/4/2012 556.2 Silva Valley WB Off-Ramp Bridge Figure 1.dwg

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.9



LEGEND

gb	Gabbro	Mesozoic
Jss	Salt Springs Slate	Jurassic
Jch	Copper Hill Volcanics	Late Middle Jurassic
um	Ultramafic rocks	Paleozoic(?)-Mesozoic(?)
ls	Limestone	Paleozoic-Mesozoic
ms	Metasedimentary	Paleozoic?
mv	Metavolcanic	Paleozoic?

Geologic Symbols

Geologic contact

Fault; solid where located, dashed where inferred.

Source: Generalized Geologic Map of El Dorado County, California by Lawrence L. Busch, June 2001.

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GEOLOGIC MAP

Silva Valley Parkway Interchange
WB Off-Ramp Bridge, EA 03-1E2901
El Dorado County, California

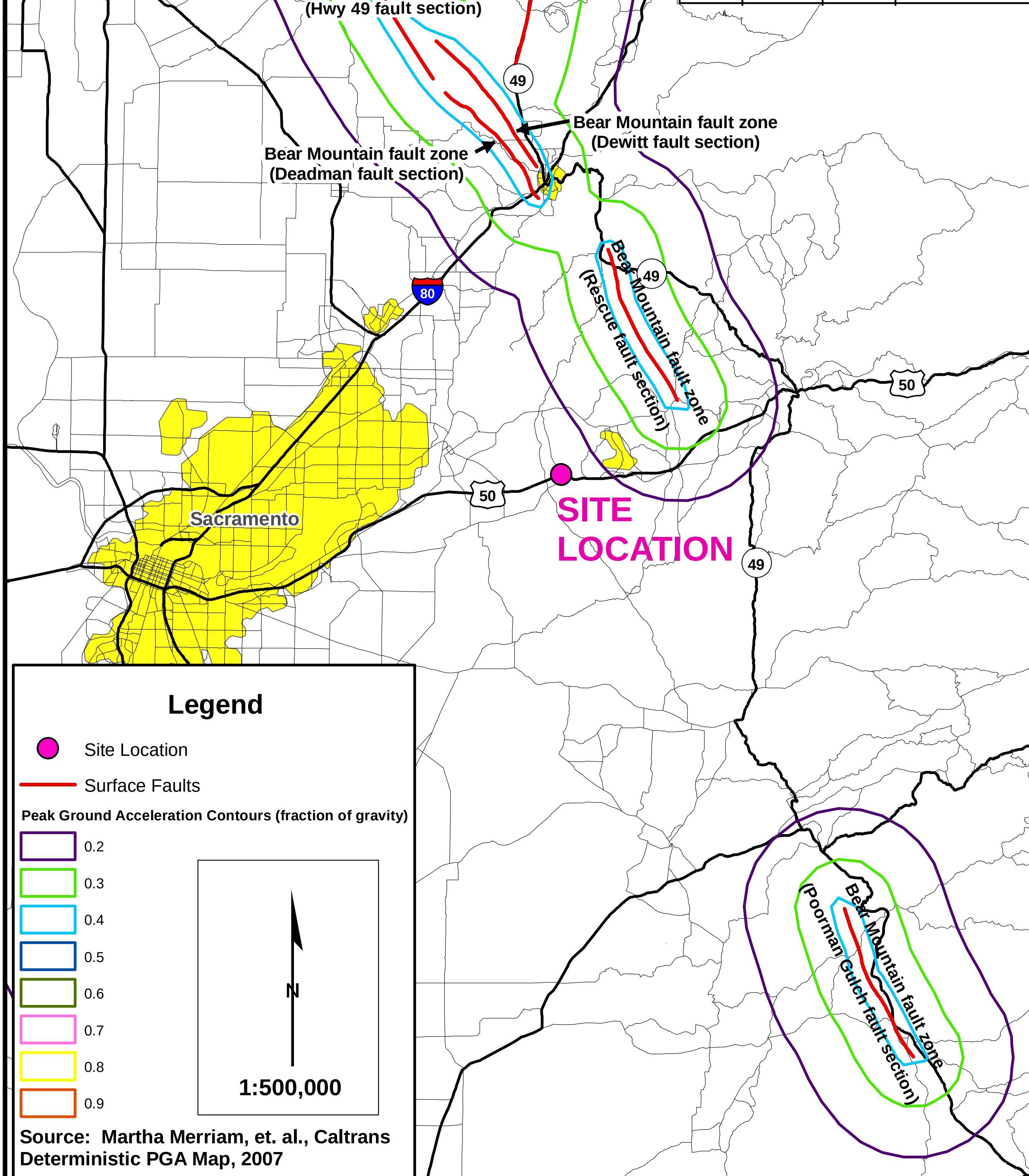
File No. 556.2

April 2012

Figure 2

5/4/2012 556.2 Silva Valley WB Off-Ramp Bridge Figure 2.dwg

Dist	County	Route	Total Project
03	ED	50	PM R1.9



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SEISMIC HAZARD MAP

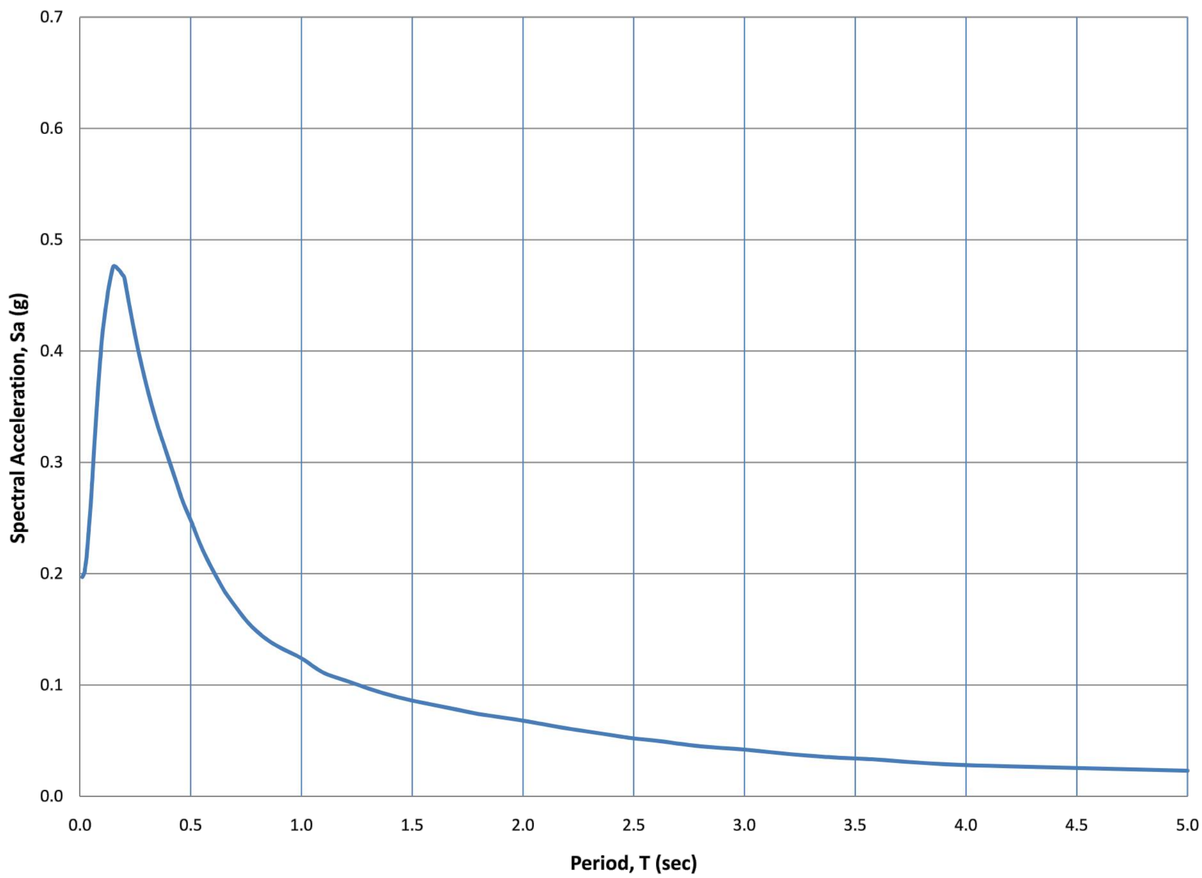
Silva Valley Parkway Interchange
WB Off-Ramp EA 03-01E2901
El Dorado County, California

File No. 556.2

April 2012

Figure 3

DIST	COUNTY	ROUTE	TOTAL PROJECT
03	ED	50	PM R1.9



Caltrans ARS Online (V1.04)

5/4/2012 556.2 Silva Valley WB Off-Ramp Bridge Figure 4.dwg

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ARS CURVE

Silva Valley Parkway Interchange
WB Off-Ramp Bridge, EA 03-1E2901
El Dorado County, California

File No. 556.2

April 2012

Figure 4

APPENDIX B

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

Appendix B

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

CERTIFIED ENGINEERING GEOLOGIST DATE

PLANS APPROVAL DATE

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

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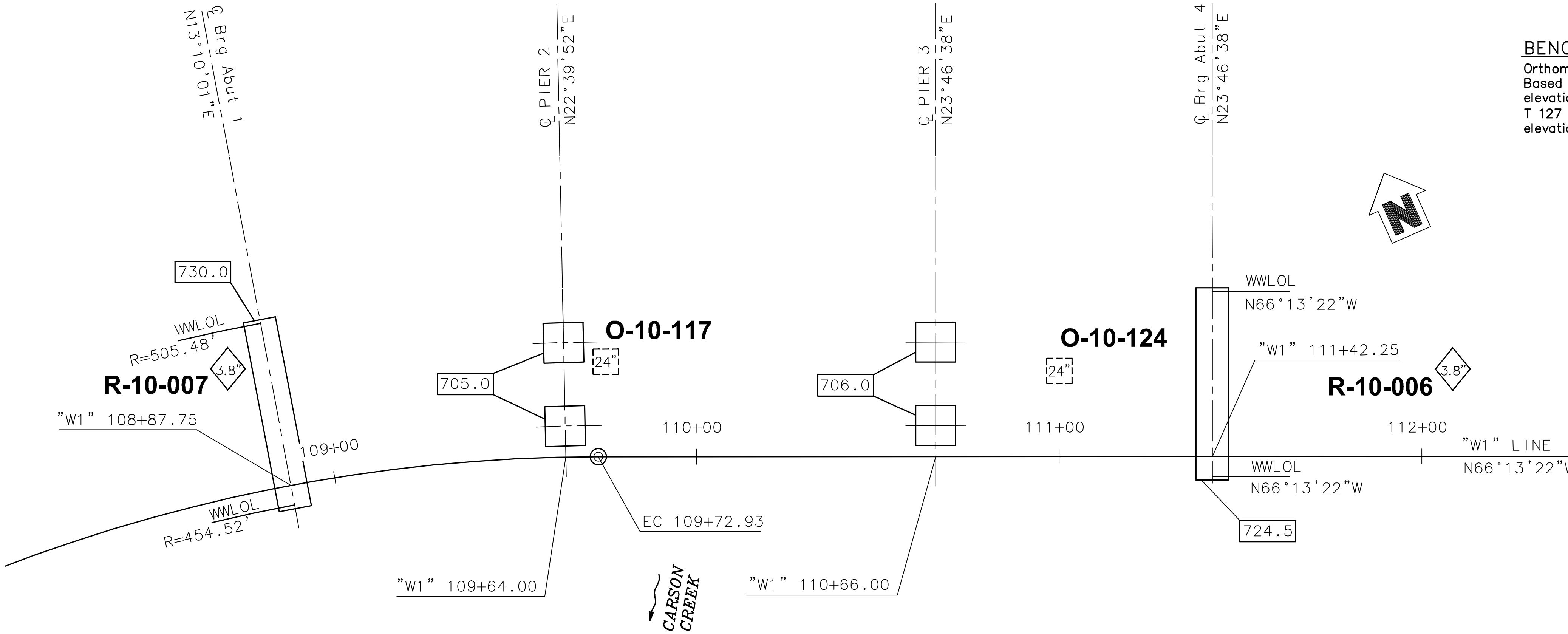
11521 BLOCKER DRIVE, SUITE 110
AUBURN, CA 95603 FILE No. 556.2

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

BENCHMARK

Orthometric Heights shown are NGVD 29 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.

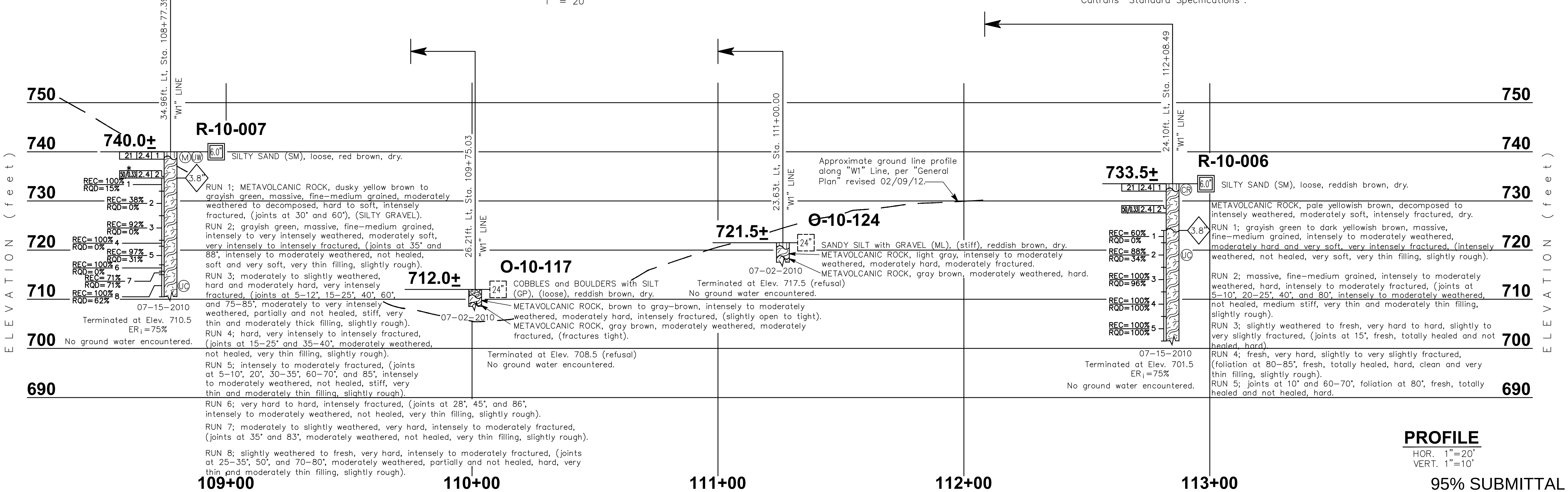


PLAN

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. 2, and 3, "Soil Legend" and 4, "Rock Legend".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with brass liners.
3. "2.4 inch sampler": ID=2.4 inch, OD=2.9 inch. Driven in same manner as SPT ("1.4 inch") sampler.
4. Where less than the 0.5 inches of penetration is achieved, the blow count shown is for that fraction of the interval actually penetrated.
5. Where indicated by an asterisk (*) the number of blows shown is for only that fraction of the initial 0.5 ft. "seating drive" interval penetrated.
6. If laboratory tests are not shown as being performed, the soil descriptions presented in the LOTB are based solely on the visual practices described in the before mentioned Manual.
7. The length of each sampled interval is shown graphically on the boring log.
8. Consistency of soils shown in ( ) where estimated.
9. Groundwater surface (GWS) reflect the fluid level in the borings on the specified date. Groundwater surface is subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
10. Electronic media for plan view provided by Mark Thomas & Co., Inc., dated April 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".



PROFILE

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

95% SUBMITTAL

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

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0130K
POST MILE R1.9

SILVA VALLEY WB OFF-RAMP BRIDGE
LOG OF TEST BORINGS 1 OF 4

GS GEOLOGIST LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 0259
PROJECT NUMBER & PHASE: 0200000258
CONTRACT NO.: 1E2901

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 04/30/12 04/30/12
SHEET 16 OF 19

FILE => $REQUEST

5/4/2012 556.2 Silva Valley WB Off-Ramp Bridge LOTB.dwg

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

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

CERTIFIED ENGINEERING GEOLOGIST   DATE

PLANS APPROVAL DATE

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AUBURN, CA 95603   FILE No. 556.2

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

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

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

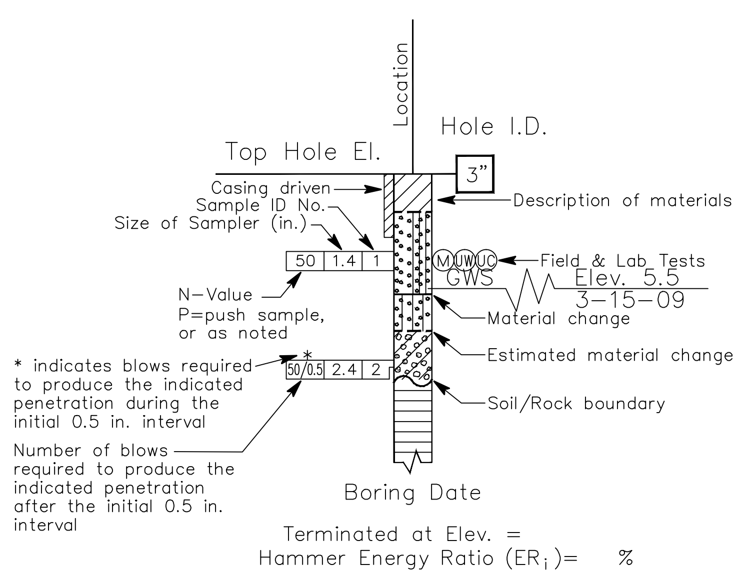
BOREHOLE IDENTIFICATION

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

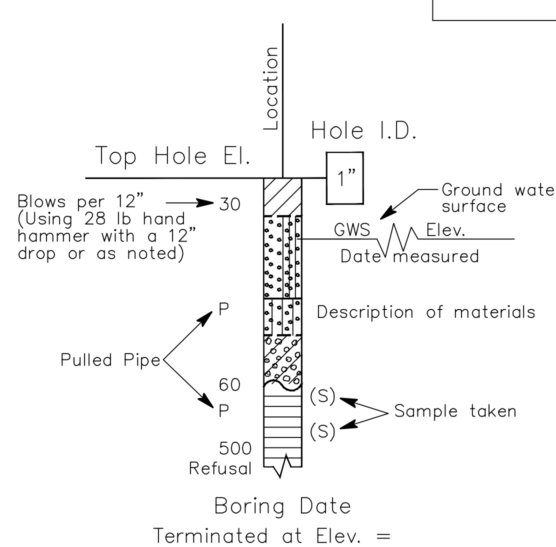
NOTE: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

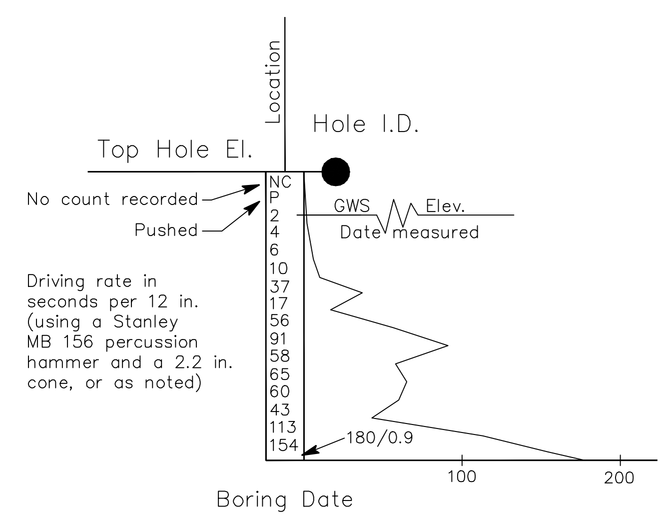
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be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



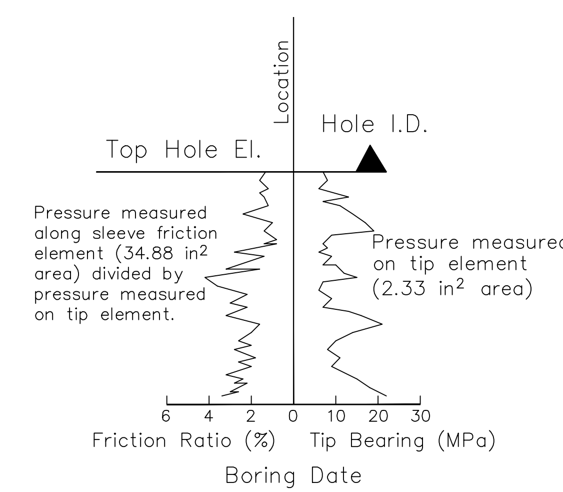
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

95% SUBMITTAL

SOIL LEGEND

SILVA VALLEY WB OFF-RAMP BRIDGE

LOG OF TEST BORINGS 2 OF 4

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. 25-0130K
POST MILE R1.9

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

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 0259
PROJECT NUMBER & PHASE: 0200000258
CONTRACT NO.: 1E2901

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES 04/18/12 04/30/12
SHEET 17 OF 19

FILE => $REQUEST

556.2 Silva Valley WB Off-Ramp Bridge LOTB.dwg
5/4/2012

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	1.06/2.90		

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

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

95% SUBMITTAL

SOIL LEGEND

SILVA VALLEY WB OFF-RAMP BRIDGE

LOG OF TEST BORINGS 3 OF 4

DESIGN OVERSIGHT

SIGN OFF DATE

DRAWN BY M. ROBERTSON

CHECKED BY R. PICKARD

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0130K
POST MILE R1.9

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

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 0259
PROJECT NUMBER & PHASE: 0200000258
CONTRACT NO.: 1E2901

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 04/30/12 04/30/12

SHEET 18 OF 19

FILE => $REQUEST

5/4/2012 556.2 Silva Valley WB Off-Ramp Bridge LOTB.dwg

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

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

CERTIFIED ENGINEERING GEOLOGIST DATE

PLANS APPROVAL DATE

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

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

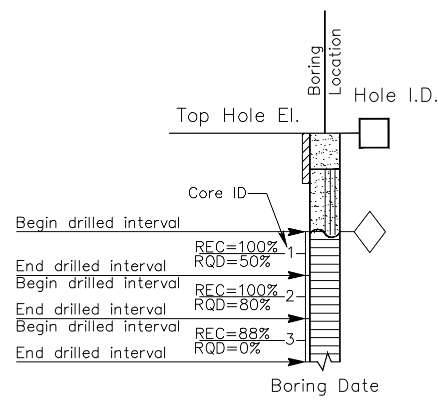
MARK THOMAS & CO., INC.
7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

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

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

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

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



RELATIVE STRENGTH OF INTACT ROCK

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

BEDDING SPACING

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

LEGEND OF ROCK MATERIALS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

ROCK HARDNESS

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

FRACTURE DENSITY

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

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

WEATHERING DESCRIPTORS FOR INTACT ROCK

Description	Diagnostic features					General Characteristics
	Chemical weathering–Discoloration and/or oxidation		Mechanical Weathering–Grain boundary conditions (disaggregation) primarily for granitics and some coarse–grained sediments	Texture and solutioning		
	Body of rock	Fracture Surfaces		Texture	Solutioning	
Fresh	No discoloration, not oxidized.	No discoloration or oxidation.	No separation, intact (tight).	No change.	No solutioning.	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull.	Minor to complete 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 hair–line fractures, or veinlets. Rock is significantly weakened.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe–Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated).	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete.		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

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

95% SUBMITTAL

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

ROCK LEGEND

SILVA VALLEY WB OFF-RAMP BRIDGE

LOG OF TEST BORINGS 4 OF 4

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

R. PICKARD
FIELD INVESTIGATION BY:
DATE: July 2010

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25–0130K
POST MILE R1.9

UNIT: 0259
PROJECT NUMBER & PHASE: 0200000258 CONTRACT NO.: 1E2901

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 04/30/12 04/30/12 SHEET 19 OF 19

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

5/4/2012 556.2 Silva Valley WB Off–Ramp Bridge LOTB.dwg

FILE => $REQUEST

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

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

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

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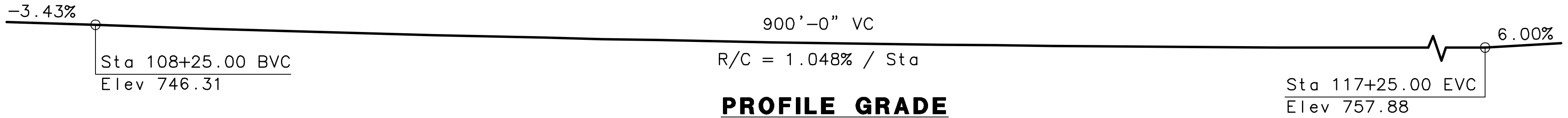
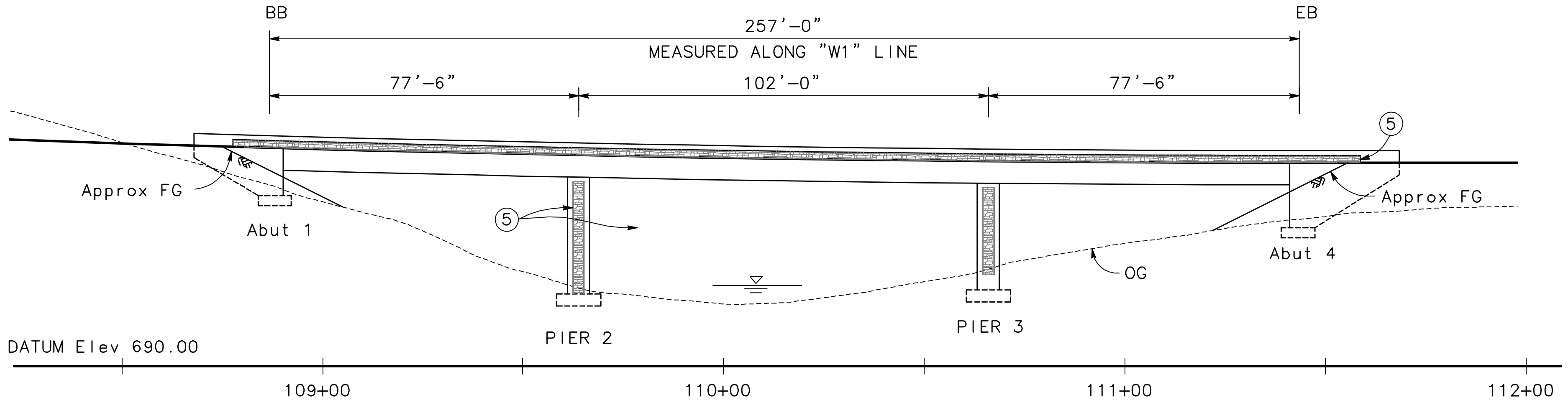
COUNTY OF EL DORADO
DEPT. OF TRANSPORTATION
4950 HILLSDALE CIR STE 400
EL DORADO HILLS, CA 95762

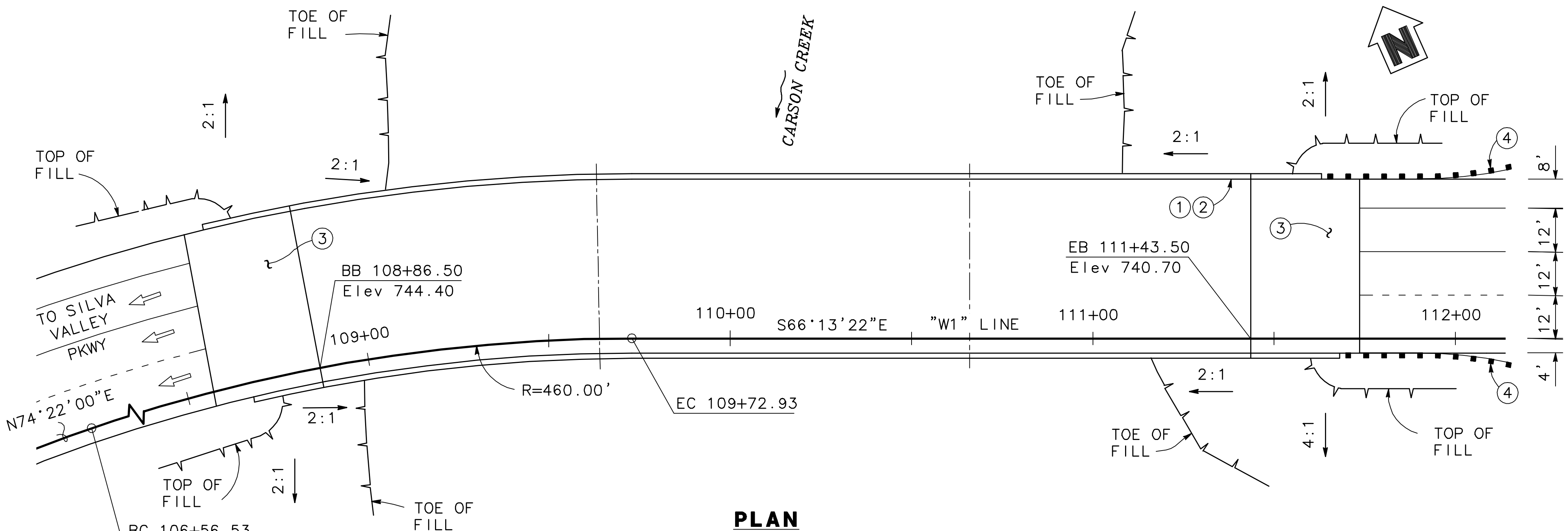
MARK THOMAS & COMPANY, INC.
7300 FOLSOM BOULEVARD, SUITE 203
SACRAMENTO, CA 95826



PROFILE GRADE
NO SCALE



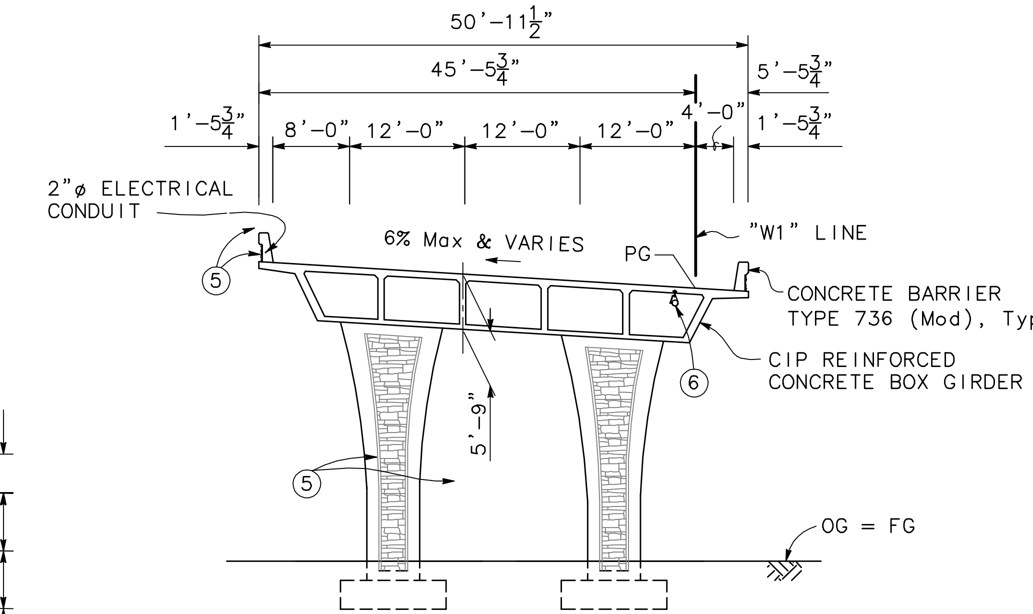
ELEVATION
1" = 20'



PLAN
1" = 20'

"W1" LINE
R = 460.00'
Δ = 39°24'38"
L = 316.41'
T = 164.75'

INDEX TO PLANS



TYPICAL SECTION
1" = 10'

NOTES:

1. Paint "SILVA VALLEY WB OFF-RAMP BRIDGE"
2. Paint "BR. NO. 25-0130K"
3. Structure Approach Slab Type N(30S)
4. MBGR, see "ROAD PLANS"
5. Dry Stack Rock Texture
6. Deck Drain

1. For GENERAL NOTES, see "DECK CONTOURS" sheet.
2. For SPREAD FOOTING DATA TABLE & HYDROLOGIC SUMMARY, see "FOUNDATION PLAN" sheet.

95% SUBMITTAL

DESIGN OVERSIGHT	DESIGN	BY T. PHAM	CHECKED V. SHERBY	LOAD & RESISTANCE FACTOR DESIGN	LIVE LOADING: HL93 W/"LOW-BOY"; PERMIT DESIGN VEHICLE
	DETAILS	BY G. BOYKO	CHECKED T. PHAM	LAYOUT BY D. MINNEMA	CHECKED T. PHAM
SIGN OFF DATE	QUANTITIES	BY	CHECKED	SPECIFICATIONS BY J. PASSALACQUA	PLANS AND SPECS COMPARED P. CHEN

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0130K
POST MILES

SILVA VALLEY WB OFF-RAMP BRIDGE
GENERAL PLAN

DESIGN GENERAL PLAN SHEET (ENGLISH) (REV.7/16/10)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 0259
PROJECT NUMBER & PHASE: 0300000258

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY): 10/28/10 03/09/12 03/21/12 01/18/11 02/14/11 03/03/11 04/06/11 04/27/11 02/09/12

SHEET 1 OF 19

CONTRACT NO.: 71328 PROJECT ID:

FILE => $REQUEST

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

V:\West Valley LLC-55-0162B-Silva Valley Pkwy IC\CADD\Structures\Westbound off-ramp\01 WB off Ramp_gp.dwg 4/9/2012 3:16 PM

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

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

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

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DEPT. OF TRANSPORTATION
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MARK THOMAS & CO., INC.
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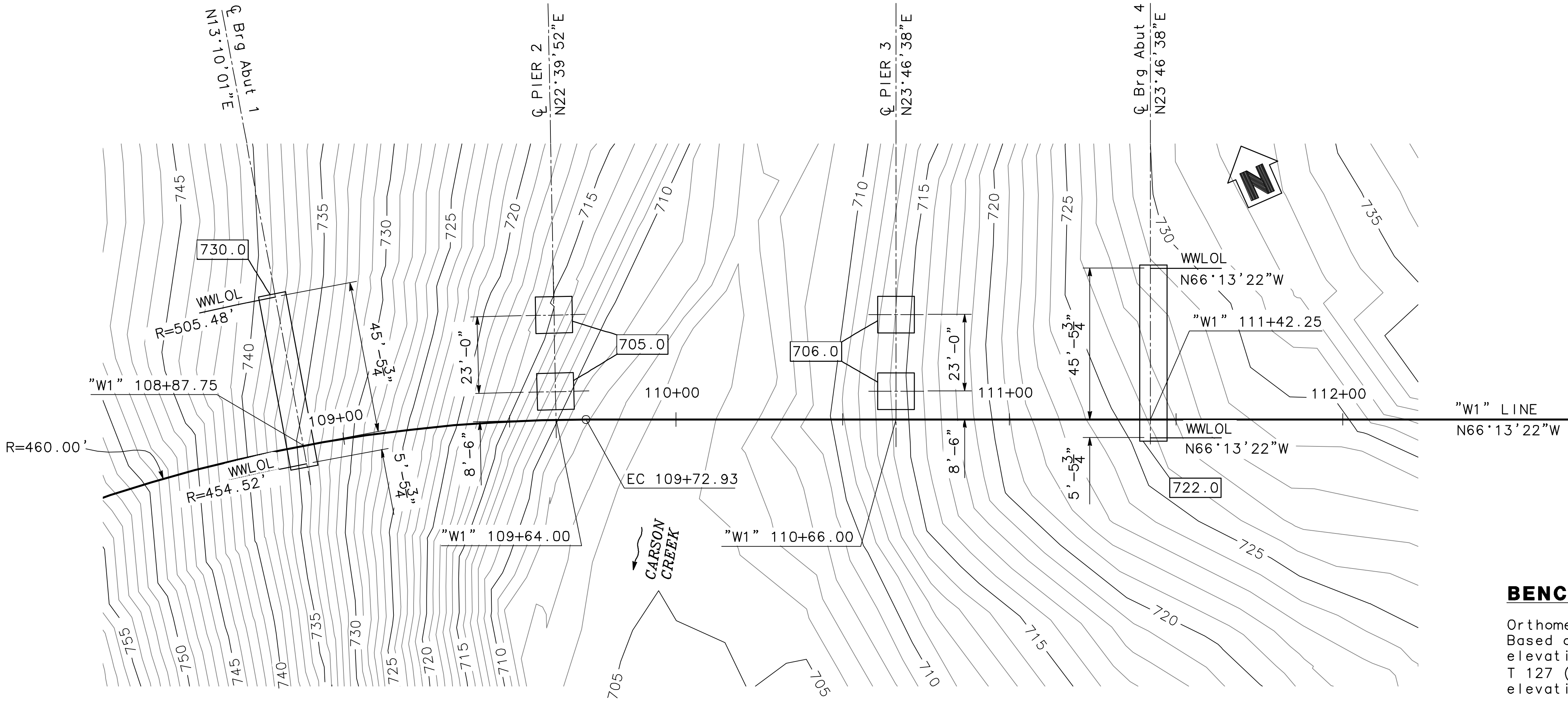


PLAN

1" = 20'

"W1" LINE

R = 460.00'
Δ = 39°24'38"
L = 316.41'
T = 164.75'

BENCHMARK

Orthometric Heights shown are NGVD 29 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.

LEGEND:

▭ Indicates Bottom of Footing Elevation (feet)

SPREAD FOOTING DATA TABLE

Support Location	Working Stress Design (WSD)		Load and Resistance Factor Design (LRFD)		
	Permissible Gross Contact Stress (Settlement) (ksf)	Allowable Gross Bearing Capacity (ksf)	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength Factored Gross Normal Bearing Resistance **φb = 0.45** (ksf)	Extreme Event Factored Gross Normal Bearing Resistance **φb = 1.00** (ksf)
Abut 1	10	10	N/A	N/A	N/A
Pier 2	N/A	N/A	20	22.5	50
Pier 3	N/A	N/A	20	22.5	50
Abut 4	10	10	N/A	N/A	N/A

HYDROLOGIC SUMMARY

Drainage Area: 3.29 Square Miles

	Design Flood	Base Flood	Overtopping Flood
Frequency (Years)	50	100	N/A
Discharge (Cubic Foot per Sec)	1853	2060	N/A
Water Surface (Elevation at Bridge)	711.37	711.60	N/A

Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.

95% SUBMITTAL

GEOTECHNICAL PROFESSIONAL APPROVAL DATE

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

DESIGN OVERSIGHT
SIGN OFF DATE

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

DESIGN BY T. PHAM CHECKED V. SHERBY
DETAILS BY G. BOYKO CHECKED T. PHAM
QUANTITIES BY CHECKED

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

JULIE PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 25-0130K
POST MILES

SILVA VALLEY WB OFF-RAMP BRIDGE

FOUNDATION PLAN

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

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

UNIT: 0259
PROJECT NUMBER & PHASE: 0200000258

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY): 08/06/10, 03/03/11, 04/06/11, 04/27/11, 02/09/12, 03/08/12, 03/19/12, 01/18/11, 02/14/11

SHEET 3 OF 19

FILE => $REQUEST

CONTRACT NO.: 71328 PROJECT ID:

APPENDIX C

Laboratory Test Results

LABORATORY TEST RESULTS

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

- Moisture Content-Dry Density (ASTM D2937 & D2216)
- Unconfined Compressive Strength – Rock (ASTM 2938)
- pH/Minimum Resistivity (CTM 643)
- Chloride (CTM 422)
- Sulfate (CTM 417)

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

Blackburn Consulting
US 50 / Silva Valley Parkway Interchange, Westbound Off-Ramp
Bridge at Carson Creek, EA 03-1E2901, PM R1.90
File No. 556.2
April 2012

Laboratory Testing Summary

Exploration I.D.	Sample No.	Depth (feet)	Sample Type	USCS Classification	Moisture Content (%)	Dry Density, γ_{dry} (pcf)	Unconfined Compression (psi)	Corrosivity			
								pH	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
R-10-006	S1	0.0-1.5	MC	ML				5.3	6970	14.0	0.2
R-10-006	Core	13.0-14.3	HQ	Rock			5,810				
R-10-007	S1	1.0-1.5	MC	ML	9.4	110					
R-10-007	Core	28.0-29.0	HQ	Rock			18,330				

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 RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : SILVA VLY PKWY INTER Site ID : R10-006-S1B.
Thank you for your business.

* For future reference to this analysis please use SUN # 58852-119540.

EVALUATION FOR SOIL CORROSION

Soil pH	5.30		
Minimum Resistivity	6.97 ohm-cm (x1000)		
Chloride	14.0 ppm	00.00140	%
Sulfate	0.2 ppm	00.00002	%

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

APPENDIX D

Calculations and Analyses

Appendix D

WB Off, Silva Valley Parkway

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

Model Inputs

Fault

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

Distance

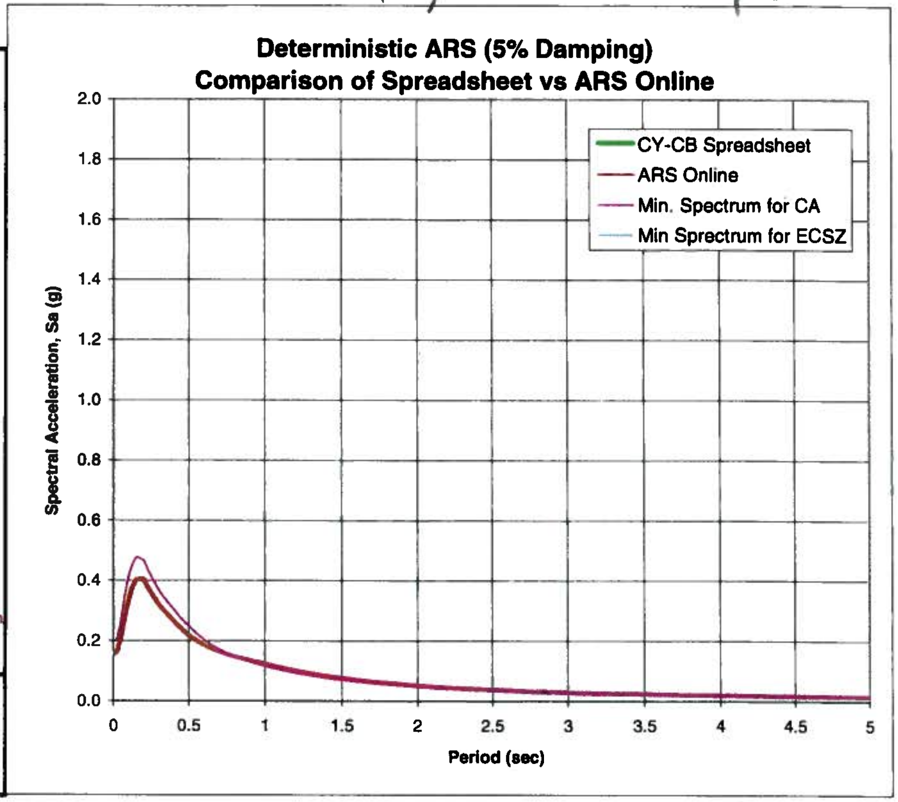
R_{RUP} (km)	12.6	
R_{JB} (km)	12.6	
R_x (km)	12.5	
Hanging Wall?	☐ Yes?	
Near-Field Factor?	☑ Yes?	

Site

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

Analysis

ARS Online vs CY-CB Spreadsheet Results
MAX. % Diff. = 2%



ARS

Page 1 of 3

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

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

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

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

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

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

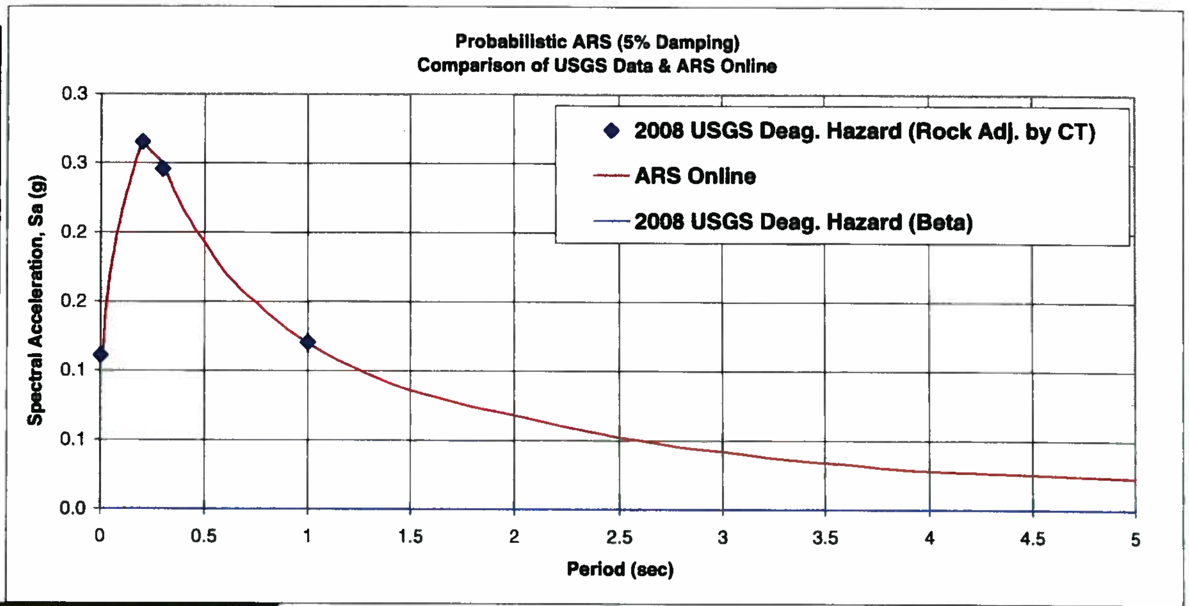
ARS

WB Off, Silva Valley Parkway

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

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

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



Place ARS Online Probabilistic Data Here "Paste"

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

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

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

Max % Difference = 1.7%

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

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

Max % Difference = 0.0%

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Abutments

For abutments, footings will be within weathered rock at depths of ~4 to 9 feet. Rock is intensely fractured at this depth and within 1B of footing base.

From BDS Figure 4.4.8.1.1A
Can assume q_{all} = **10 tsf**

Check using equation 4.4.8.1.2-1 (for jointed rock)
$q_{ult} = N_{ms} C_o$ $N_{ms} = 0.024$ $C_o = 6{,}000$ psi $= 864$ ksf
$= 0.024 (864 \text{ ksf}) = 20.7$ ksf $=$ **10.4 tsf**
(Note: RMR = 41, Poor to fair rock, Class III–IV)
$q_{all} = 10.4 \text{ tsf}/3 = 3.5 \text{ tsf} = 7$ ksf
Use q_{all} = **10 ksf** for WSD *

Piers

For Piers, test pits encountered refusal at 3-4 feet on strong rock that is moderately fractured. Less weathered and fractured rock is expected at pier locations based on lower elevation (rock more recently eroded), test pits, outcrop, exposures in creek bed (base will be below creek bed) and core results at abutments.

Conservative estimate of RQD = 50%
From BDS Figure 4.4.8.1.1A
Can assume q_{all} = **+50 tsf**
Use conservative q_{ult} = 25 tsf = **50 ksf**

For LRFD
Strength $q_R = 0.45 (50 \text{ ksf}) = 22.5$ ksf
Extreme $q_R = 1.0 (50 \text{ ksf}) = 50.0$ ksf

* For q_{all} also see attached sheet (8 of 8) with evaluation as a soil with $\phi = 35°$. $q_{all} = 11.4$ ksf at Abuts.

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Subject Foundation Settlement	File # 556 By PFF	Date 11/3/10

rev 3/12/12

Abutments

Abutments will be on fractured but "competent" rock. Per BDS 4.4.8.2.1, can assume settlement will be less than 1/2 inch

Check for footing on jointed rock (BDS 4.4.8.2.2)

$\rho = q_o(1-v^2) B I_p / E_m$, $I_p = (53/8.25)^{1/2} / 1.29 = 1.97$

(note B_z from Table 4.4.7.2.2B)

$q_o = \sim 6$ ksf but use 10 ksf to check

$v = 0.22$ $B = 8.25$ feet $E_m = 0.15(5.0 \times 10^6) =$

(note for E_m see eq. 4.4.8.2.2-3) $= 0.75 \times 10^6$ psi $= 1.08 \times 10^5$ ksf

$\rho = 10(1-0.22^2) 8.25(1.97) / 108{,}000$ ksf

$= 9.516$ ksf $(16.25\,ft) / 108{,}000$ ksf

$= 154.64$ kft $/ 108{,}000$ ksf $= 1.432 \times 10^{-3}$ ft $=$ .02 inches

Settlement is less than 1/2" at 10 ksf loading

For WSD $q_{ps} =$ 10 ksf plus

Piers

Competent rock at pier locations. Can assume settlement less than 1/2 inch

For LRFD $q_{pn} =$ 20 ksf plus

RDS

or open not wider than $^1/_8$ inch. For footings on less competent rock, more detailed investigations and analyses should be used to account for the effects of weathering, the presence and condition of discontinuities, and other geologic factors.

4.4.8.1 Bearing Capacity

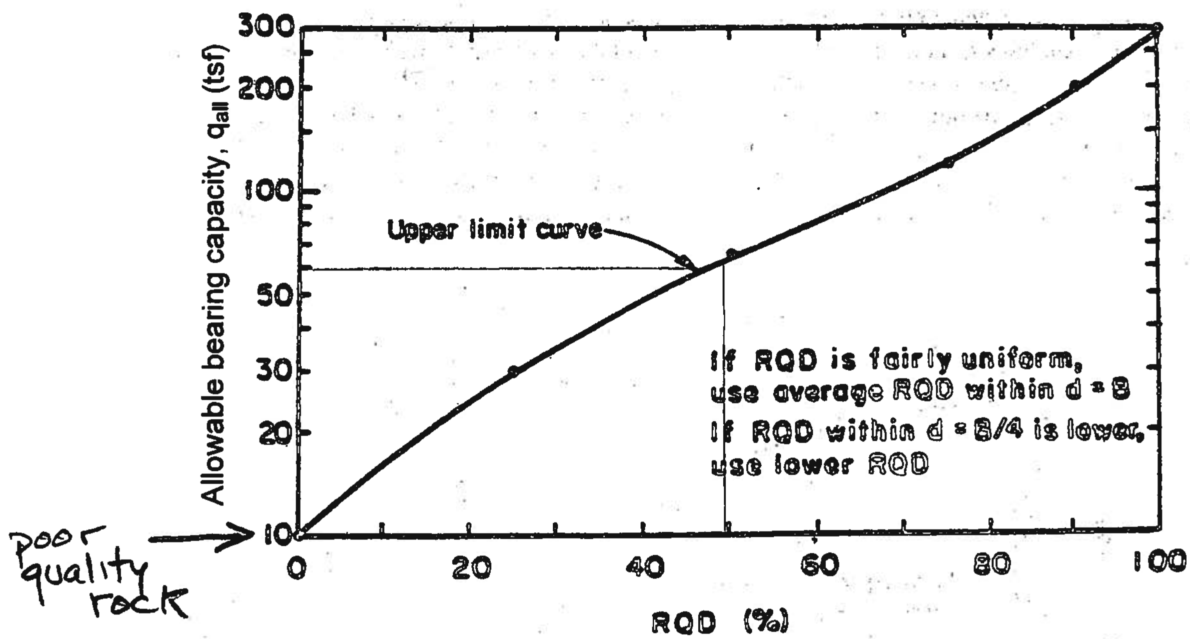
4.4.8.1.1 Footings on Competent Rock

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

4.4.8.1.2 Footings on Broken or Jointed Rock

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

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



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

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

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

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

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

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

4.4.8.1.3 Factors of Safety

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

4.4.8.2 Settlement

4.4.8.2.1 Footings on Competent Rock

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

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800- 45,000
		Limestone	500- 6,000	3,500- 42,000
		Carbonatite	800- 1,500	5,500- 10,000
		Marble	800- 5,000	5,500- 35,000
		Tactite-Skarn	2,700- 7,000	19,000- 49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200- 21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600- 28,000
		Phyllite	500- 5,000	3,500- 35,000
		Siltstone	200- 2,500	1,400- 17,000
		Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000- 30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800- 32,000
		Sandstone	1,400- 3,600	9,700- 25,000
		Quartzite	1,300- 8,000	9,000- 55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000- 26,000
		Diabase	450-12,000	3,100- 83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000- 40,000
		Gabbro	2,600- 6,500	18,000- 45,000
		Gneiss	500- 6,500	3,500- 45,000
		Granite	300- 7,000	2,100- 49,000
		Quartzdiorite	200- 2,100	1,400- 14,000
		Quartzmonzonite	2,700- 3,300	19,000- 23,000
		Schist	200- 3,000	1,400- 21,000
		Syenite	3,800- 9,000	26,000- 62,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations.
[2]Not including oil shale.

4.4.8.2.2 *Footings on Broken or Jointed Rock*

Where the criteria for competent rock are not met, the influence of rock type, condition of discontinuities and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock may be determined using the following:

- For circular (or square) footings;

$$\rho = q_o\,(1-\nu^2)rI_\rho/E_m,\ \text{with}\ I_\rho = (\qquad)/\beta_z \qquad (4.4.8.2.2\text{-}1)$$

- For rectangular footings;

$$\rho = q_o\,(1-\nu^2)BI_\rho/E_m,\ \text{with}\ I_\rho = (L/B)^{1/2}/\beta_z \qquad (4.4.8.2.2\text{-}2)$$

Values of I_ρ may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

TABLE 4.4.8.2.2A Summary of Poisson's Ratio for Intact Rock Modified after Kulhawy (1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν Maximum	Minimum	Mean	Standard Deviation
Granite	22	22	0.39	0.09	0.20 Similar rock	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

use ν= 0.22

TABLE 4.4.8.2.2B Summary of Elastic Moduli for Intact Rock Modified after Kulhawy (1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_o (psi x 10^6)(1) Maximum	Minimum	Mean	Standard Deviation
Granite	26	26	14.5	0.93	7.64 Similar rock	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.80	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.70	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

(1) 1.0×10^6 psi = 1.44×10^5 ksf.

use E_o= 5.0 as conservative estimate

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231\,(RQD) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

+ *4.4.8.2.3 Deleted*

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on or near a slope by limiting equilibrium methods of analysis which employ the Modified Bishop, simplified Janbu, Spenser or other generally accepted methods of slope stability analysis. Where soil and rock parameters and ground water levels are based on in-situ and/or laboratory tests, the minimum factor of safety shall be 1.3 (or 1.5 where abutments are supported above a slope). Otherwise, the minimum factor of safety shall be 1.5 (or 1.8 where abutments are supported above a retaining wall).

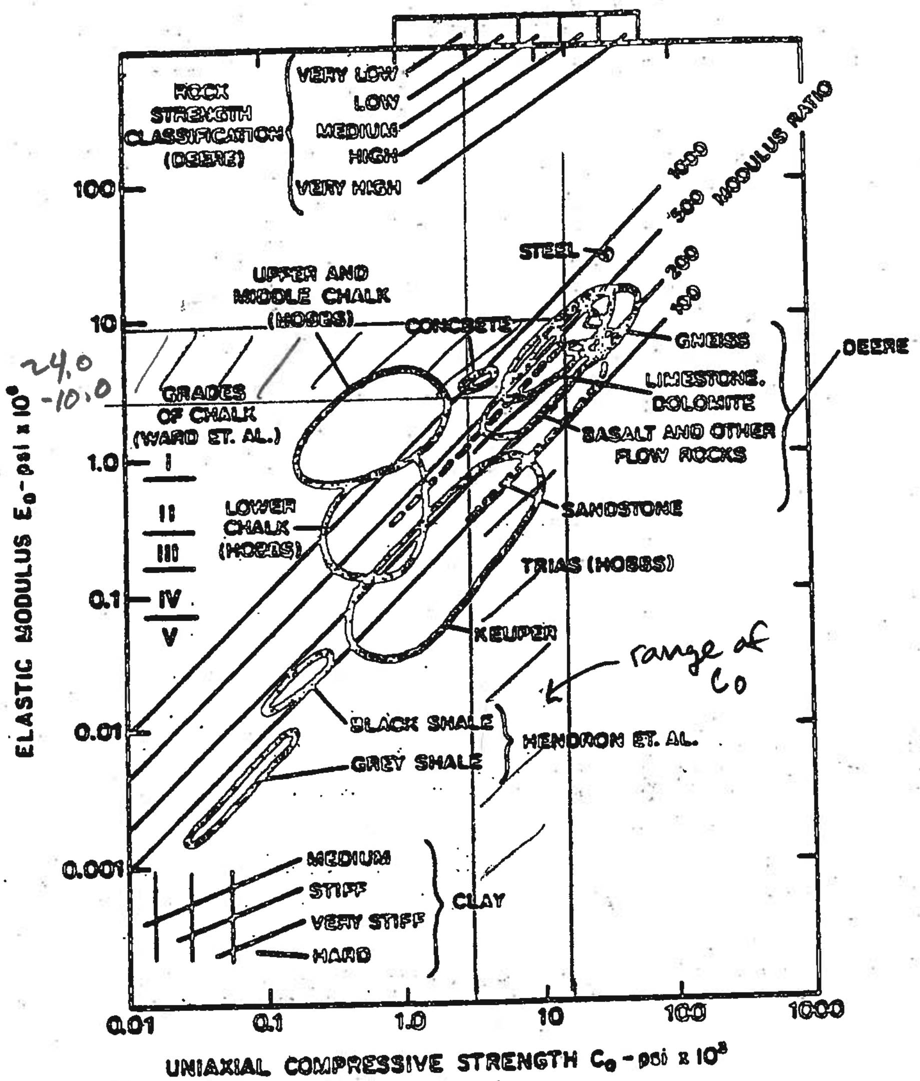


FIGURE 4.4.8.2.2A Relationship Between Elastic Modulus and Uniaxial Compressive Strength for Intact Rock Modified after Deere (1968)

pg 7 of 8

BEARING CAPACITY – STRENGTH LIMIT STATE (AASHTO Bridge Design Specifications)

Date: 3/12/2012
Project: Silva Valley WB Off-Ramp
BCI No: 556.2

Support: **Abut 1 & 4, underlain by fractured/weathered rock**
Boring: R-10-007 and R-10-006
Note: ***CHECK with rock capacity modeled as a soil***

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)

$N_c, N_q, and\ N_\gamma$ = bearing capacity factors

C_{wq} & $C_{w\gamma}$ = correction factors for location of ground water

$s_c, s_\gamma, and\ s_q$ = footing shape correction factors

d_q = correction factor to account for shearing resistance in material above bearing level

$i_c, i_\gamma, and\ i_q$ = load inclination factors

D_w = depth to ground water taken from the ground surface (feet)

Input Parameters

γ =	135	(pcf)
ϕ =	35	(degrees)
c =	0	(psf)
D_f =	4	(feet)
D_w =	19	(feet)

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

Bottom Footing Elevation (feet):	730.0
Finished Grade (feet):	734.0
Ground Water Elevation (feet):	715.0

Resistance Factor (φ_b)= 0.33

Bearing Capacity Factors

$N_c = (N_q - 1)\cot\phi = 46.1$

$N_q = \exp(\pi\tan\phi)\tan^2(45 + \phi/2) = 33.3$

$N_\gamma = 2(N_q + 1)\tan\phi = 48.0$

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

Solve for Ultimate Gross Bearing Capacity

Effective Footing Dimensions							Ultimate Gross Bearing Capacity			Strength Limit State Allowable Gross Bearing Capacity (Factor of Safety = 3.0)		
B'	L'	C_{wq}	$C_{w\gamma}$	s_c	s_γ	s_q						
(feet)							(psf)	(ksf)	(tsf)	(psf)	(ksf)	(tsf)
5.0	53.0	1.00	1.00	1.00	1.00	1.00	34182	34.18	17.1	11393	11.39	5.7

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)

EQUIVALENT FLUID WEIGHTS (EFWs)

Lateral Pressures
Pg 1 of 1

Project: Silva Valley Parkway Interchange, WB Off-Ramp Bridge
BCI No.: 556.2
Date: 11/4/2010
By: PFF

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

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

EFW = Kγ

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

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

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

Static Loading

Active Pressure Coefficient (K_A):

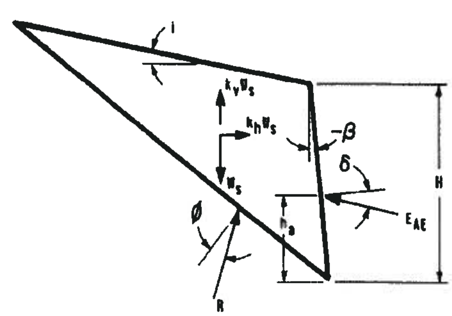
$$K_A = [\cos\phi/\{1 + [\sin\phi(\sin\phi - \cos\phi\tan i)]^{0.5}\}]^2$$

Passive Pressure Coefficient (K_P):

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

At-rest Pressure Coefficient (K_O):

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



Seismic Loading

Seismic Active Pressure Coefficient (K_{AE}):

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

Seismic Passive Pressure Coefficient (K_{PE}):

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

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

* Level Backfill Condition Only.

APPENDIX E

Draft Report Comment and Response

OGDN Review Comment & Response Form

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

Project Name:
Silva Valley Pkwy Interchange

Liaison Engineer:
Erick Fredrickson</td><td colspan="2">☐ 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:

Structure Information</td><td rowspan="2">**Reviewer:** Thomas Song, PE
Functional Unit: 59-323 (Geotech North)
EFIS: 59-3657
Phone Number: (916) 227-1057
e-mail: Thomas_song@dot.ca.gov
Date of Review: 12/3/2010</td></tr>
<tr><td>**Structure Name**
Silva Valley Pkwy OC
EB Off-Ramp UC
WB On-Ramp UC
WB Off-Ramp Br
WB Off-Ramp Retaining Wall
Carson Creek MSE Wall
Bucks Ravine Creek RCB</td><td>**Bridge No.**
25-0127
25-0128S
25-0129K
25-0130K</td></tr>
</table>

Consultant Information (to be filled in by Consultant)

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

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

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

✓= Comment Resolved
(for Reviewer's use)

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

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

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
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(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.	Not applicable to the West bound off-ramp	
14	Silva Valley Westbound On-Ramp UC, Br. No. 25-0129K, Draft Foundation Report, Appendix D, Design Calculation, Slope Stability Output	What groundwater condition is considered in the slope stability analyses? This comment applies to some other structures too.	Not applicable to the West bound off-ramp	
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?	The split was for geotechnical design due to ground slope. This is changed in final design.	
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.	This is intended as a depth from the final ground surface and has been clarified in the report.	
15	Silva Valley Westbound Off-Ramp Retaining Wall, General Plan No. 1, Sheet 1 of 6	The plan indicates there'd be more than 5 feet excavation to construct the wall footing, which may require temporary shoring. This comment applies to Carson Creek MSE Wall too.	Not applicable to the FR for the West bound off-ramp	
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."	Not applicable to the West bound off-ramp	

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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.		

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(for Reviewer's use)

Office of Special Funded Projects
Comment & Response Form

(Revised 12/01/09)

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

Consultant Information (to be filled in by Consultant)

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

#	Doc. (See Note 1)	Page, Section, or SSP	ADDITIONAL FOUNDATION REPORT Review Comments	Consultant Responses	✓
1	**FR #25-0127**	Cover Pg	Revise "EA" to "03-1E290**1**"	Not applicable to the West bound Off-Ramp	
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.	Not applicable to the West bound Off-Ramp	
3		Pg 7	9.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the West bound Off-Ramp	
4		Pg 9	10, bullets – Verify / update all bridge information w/ final plans.	Not applicable to the West bound Off-Ramp	
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?	Not applicable to the West bound Off-Ramp	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
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✓= Comment Resolved
(for Reviewer's use)

Submittal Data (Reviewer to complete)

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

Br No*.____ *=if applicable

6	**FR #25-0129K**	Cover Pg	Include PM. Revise "EA" to "03-1E290**1**"	Not applicable to the West bound Off-Ramp	
7		Pg 1	2.1 – Revise the description from "100' south" to "xx' north". Be clear between 'old / existing' and 'new' Silva Valley Parkway.	Not applicable to the West bound Off-Ramp	
8		Pg 2	1st para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2nd para – Delete 1st & 2nd sentence. 2.2, 3rd para – Verify side slopes at abutment 4 (4:1?).	Not applicable to the West bound Off-Ramp	
9		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the West bound Off-Ramp	
10		Pg 10	12 – Revise "EB Off-Ramp" with "WB On-Ramp"	Not applicable to the West bound Off-Ramp	
11	**FR #25-0128S**	Cover Pg	Include PM. Revise "EA" to "03-1E290**1**"	Not applicable to the West bound Off-Ramp	
12		Pg 1	2.1 – Be clear between 'old / existing' and 'new' Silva Valley Parkway.	Not applicable to the West bound Off-Ramp	
13		Pg 2	1st para – Include "Br. No. 25-0072" when identifying the existing Clarksville UC. 2nd para – Delete 1st & 2nd sentence.	Not applicable to the West bound Off-Ramp	
14		Pg 8	10.2 – Provide 'Mmax' used for ARS curve.	Not applicable to the West bound Off-Ramp	
15		Pg 10, 11	Table 3, 4, 5 – Revise / update footing 'L' dimension.	Not applicable to the West bound Off-Ramp	
16	**FR #25-0130K**	Cover Pg	Revise "EA" to "03-1E290**1**"	EA number is corrected	
17		Pg 2	2.2 – Revise bridge width dimension. 3 – Complete the description of the borings ("two…"borings?). Are there also "two" test pits?	2.2: Width dimension is corrected to current GP. 3: The description is completed – there were two test pits at this location.	
18		Pg 6	9.2 – Provide 'Mmax' used for ARS curve.	9.2: Mmax of 6.5 is provided.	
19		Pg 8, 9	10 – Revise / update abutment and bent footing dimensions.	10: Footing dimensions are updated to the current foundation plan.	
20					
21	Ret Wall #3	General	Can this wall be eliminated with only slope excavation? R/W is available and existing side slopes are fairly steep with rocky material.	Not applicable to the West bound Off-Ramp	
22					

23					
24					