FOUNDATION REPORT

DEAD HORSE SLOUGH BRIDGE (WIDEN)

BRIDGE NO. 12-0135
03-BUT-32, PM 11.08
EA: 1E4901; CU: 03

Prepared by:

BLACKBURN CONSULTING
2491 Boatman
West Sacramento, CA 95691
(916) 375-8706

Prepared for:

Mark Thomas and Company, Inc.
7300 Folsom Blvd., Suite 203
Sacramento, CA 95826

September 22, 2010

West Sacramento Office:
2491 Boatman Ave • West Sacramento, CA 95691
(916) 375-8706 • Fax: (916) 375-8709

blackburn consulting

Main Office: (530) 887-1494
11521 Blocker Drive, Suite 110 • Auburn, CA 95603
West Sacramento Office: (916) 375-8706

Geotechnical • Construction Services • Forensics

File No. 1202.1
September 22, 2010

Mr. Lance Schrey, P.E.
Mark Thomas & Co., Inc.
7300 Folsom Blvd., Suite 203
Sacramento, CA 95826

Subject: **FOUNDATION REPORT**
Dead Horse Slough Bridge (Widen), Bridge No. 12-0135
03-BUT-32, PM 11.08, CU: 03, EA: 1E4901
Butte County, California

Dear Mr: Schrey

Blackburn Consulting (BCI) prepared this Foundation Report for the Dead Horse Slough Bridge (Widen) in Butte County, California. BCI prepared this report in accordance with our June 5, 2009 Agreement between BCI and Mark Thomas & Company, Incorporated.

Thank you for the opportunity to be part of your design team. Please call if you have questions or require additional information.

Sincerely;

BLACKBURN CONSULTING

W. Eric Nichols, C.E.G.
Senior Project Manager

Reviewed by:

Rick Sowers, P.E., C.E.G.
Senior Project Manager

Distribution: Six (6) to Mark Thomas & Company, Inc., Attn: Mr. Lance Schrey, P.E.

FOUNDATION REPORT
Dead Horse Slough Bridge (Widen), Bridge No. 12-0135
03-BUT-32, PM 11.08, CU: 03, EA: 1E4901
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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Foundation Report for the Dead Horse Slough Bridge (Widen) on State Route (SR) 32 in Butte County, California. This report contains our subsurface findings, conclusions and recommendations for design of bridge foundations.

1.2 Scope of Services

To prepare this report, BCI:

1. Discussed the project with MTCo.
2. Prepared a Preliminary Foundation Report dated March 11, 2010.
3. Attended the April 7, 2010 Type Selection Meeting with the design team and Caltrans.
4. Reviewed 1957 "As-Built" plans for the existing bridge structure (Bridge No. 12-135).
5. Reviewed preliminary plans for the widen structure prepared by MTCo.
6. Reviewed published maps and literature related to site geologic and seismic conditions.
7. Observed, logged and sampled two exploratory test borings to a maximum depth of about 80 feet (ft).
8. Performed laboratory tests on samples obtained from the exploratory borings.
9. Performed engineering analysis and calculations to develop our conclusions and recommendations.
10. Prepared a Draft Foundation Report dated August 10, 2010.
11. Incorporated our responses to Caltrans Foundation review comments to the Draft Foundation Report (summarized in Appendix F).

1.3 Site Description

The project is located on SR 32, approximately one mile east of SR 99 and about 500 ft east of Forest Avenue, in Chico, California. Site coordinates are approximately latitude 39.740ºN and longitude 121.803ºW. The bridge widening is part of a 2.6 mile widening project of SR 32. We show the bridge location and project limits on Figure 1.

At this location, Dead Horse Slough flows southerly within a roughly 60-foot-wide (bottom width) channel section. Natural ground (bank) elevation at the site is approximately 242 ft above mean sea level. The existing SR 32 grade is at about this level and site topography is relatively flat. The bottom of low channel is at/near elev. 231, about 11 feet below the existing bridge deck grade. The existing banks are at about 2H:1V (horizontal:vertical distance) or flatter. The channel was dry during our review of the site in August 2009 and contained about one foot of water during our April 2010 field exploration.

The existing bridge is a 4-span, flat slab, concrete bridge, approximately 124 ft long and 32 ft wide. The bridge is supported on diaphragm abutments and 7-column bents (skewed 45° to match the channel). Existing pile foundations consist of driven step-tapered steel shells, filled with concrete, with 8-inch tip diameter and 15½-inch butt diameter. Design loads are 32 tons per pile.

1.4 Project Description

The project will widen the existing structure on the north side of the existing bridge with similar construction (cast-in-place, reinforced concrete slab). The new widened section will be 48 feet – 4 inches wide, for a total bridge width of 80 ft-10 inches. The substructure will consist of diaphragm abutments with short cantilever wingwalls and three, 10-column piers oriented to match the existing piers.

No channel modifications, other than Rock Slope Protection (RSP) at the abutments, are planned for this project. New embankment fill at the bridge approaches will be nominal (5 feet or less).

2 SUBSURFACE EXPLORATION

2.1 Caltrans

The As-Built Log of Test Borings (LOTB) drawing for the existing bridge shows three dynamic cone penetrometer borings, drilled in October, 1956, extended to depths of about 45 ft at the abutments and about 38 ft in the channel. The As-Built LOTB shows the channel bottom at about 228 ft. The dynamic cone penetrometer borings were driven to effective refusal at the indicated depths using a No. 2 McKiernan-Terry air hammer at 115 psi.

2.2 BCI Exploration

BCI retained Taber Drilling to drill and sample two exploratory borings at the site to characterize the subsurface conditions and obtain samples for laboratory testing. The drillers used a CME-55 track-mounted rig to drill the borings on April 8 and 9, 2010 to a maximum depth of about 80 ft below the ground surface (bgs). Taber used 4-inch O.D. solid flight auger, 8-inch O.D. hollow stem auger, and 3.5-inch rotary wash methods within overburden soils to relatively competent bedrock, and then switched to 3.5-inch (90 mm) HQ wireline diamond-core equipment to complete the borings. BCI determined boring locations and elevations using topography and elevation data provided by MTCo.

The drillers obtained drive samples with a Modified California sampler (equipped with 2.4-inch I.D. brass liners) and a Standard Penetration Test Sampler (1.4-inch I.D.). The samplers were driven into the ground with a 140 pound automatic trip hammer falling 30 inches. The N-values shown on the Log of Test Borings in Appendix A are uncorrected “field” values. For the Modified California sampler, BCI multiplied the field N-value by a factor of 0.65 to obtain and approximate SPT N-value.

FHWA's Soil and Foundations Reference Manual, Volume 1 (FHWA NHI-06-088, December 2006) indicates that the hammer energy transfer ratio ranges between 80-100% for automatic trip hammers. For this project, BCI assumed a hammer energy transfer ratio of 80% in the absence of recent hammer calibration data.

BCI's project engineer, Ms. Kristy Chapman, logged the borings consistent with the Unified Soil Classification System (USCS), and noted the degree of weathering, fracture density, hardness percent recovery and Rock Quality Designation (RQD) for the recovered rock cores. BCI retained soil samples recovered with the drive samplers in moisture-proof containers for laboratory testing and reference. BCI retained the rock cores in boxes. BCI also made ground water observations in the borings during and at completion of drilling operations. At the completion of field work, the borings were backfilled with cement-grout.

3 LABORATORY TESTING

BCI completed the following laboratory tests on representative soil samples obtained from the exploratory borings:

- Moisture Content - Dry Density
- Unconfined Compressive Strength
- Particle Size Analysis
- Plasticity Index and Liquid Limit
- Sulfate/Chloride Content
- pH/Minimum Resistivity

BCI performed laboratory tests in conformance with current ASTM and Caltrans test procedures. We present the laboratory test results in Appendix B.

4 GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located along the eastern side of the Great Valley Geomorphic Province, near the margin with the Sierra Nevada foothills. The Great Valley province is bounded by the Sierra Nevada to the east, the Coast Ranges to the west, the Klamath Mountains and Cascade Range to the north. The Great Valley is a broad, elongated, northwest trending, structural trough that has been filled with a thick sequence of sediments. The eastern margin of the valley is formed by the west sloping Sierran bedrock surface that extends westward beneath the alluvium and older sedimentary bedrock within the valley.

4.2 Local Geology

Mapping by Saucedo and Wagner (1992) and Helly and Harwood (1985) shows the site underlain by sediments of the Pleistocene-age Modesto formation. These sediments are primarily alluvium comprised of sand, silt and clay, and are underlain by older Pleistocene sediments of the Red Bluff formation. Bedrock of the Tuscan formation, comprised of Pliocene volcanic mudflows (lahars) with interbedded volcanic conglomerate and sandstone, underlies the Red Bluff formation. We present a Geologic Map as Figure 2.

4.3 Subsurface Conditions

4.3.1 Soil and Rock

4.3.1.1 Caltrans

The three dynamic cone penetrometer borings shown on the As-Built LOTB drawing for the existing bridge suggest relatively loose sediments to about 8 ft below channel bottom (to about elev. 220 ft), underlain by variable dense and stiff soils to about 35 ft below channel bottom. At about 38 ft below channel bottom (approximate elev. 190 ft), a consistent, very dense horizon was encountered that achieved nominal penetration using an air hammer with 115 psi driving force.

4.3.1.2 BCI Exploration

The subsurface conditions encountered in our borings are consistent with those shown on the As-Built LOTB for the existing bridge. BCI encountered three units in the test borings, as summarized below.

Unit 1: This unit extends from ground surface to a depth of about 24 feet (elev. 213 ft) in Boring A-10-B1 and 24.5 feet (elev. 215 ft) in Boring A-10-B2. It consists of layers of stiff to very stiff (locally very soft and hard) clay, sandy clay, silt, and silt with sand and dense to very dense clayey gravel, sand with gravel, and gravel with clay. We interpret Unit 1 soil as alluvium of the Modesto (or younger) formation.

Unit 2: This unit underlies Unit 1 soils and is comprised of very stiff (locally medium stiff and hard) silt and clay with varying amounts of sand and medium dense to very dense clayey sand and gravel with sand and clay. We encountered this unit to a depth of about 55 feet (elev. 182 ft) in Boring A-10-B1 and 47 feet (elev. 192 ft) in Boring A-10-B2. We interpret these soils as associated with the Red Bluff formation.

Unit 3: This unit underlies Unit 2 soils and is comprised of variably weathered and fractured sedimentary rock. We encountered this unit to the full depth of exploration -- a depth of about 80 feet (elev. 157 ft) in Boring A-10-B1 and 60 feet (elev. 179 ft) in Boring A-10-B2. We interpret these materials as Tuscan formation bedrock.

Refer to the Log of Test Borings (LOTB) and As-Built LOTB in Appendix A for more specific soil descriptions, sampling methods, laboratory test results, and blow count data. We include the required LOTB Sheet Checklist in Appendix A.

4.3.2 Ground Water

Table 1 presents the ground water depths/elevations measured in our borings during drilling.

Table 1: Ground Water Elevations

Boring Number	Reading Date	Ground Water Depth (ft)	Ground Water Elevation (ft)
A-10-B1	4/9/2010	10.6	226.4
A-10-B2	4/8/2010	10.0	229.0

The 1957 As-built LOTB shows ground water in Boring B-1 at elev. 223.5 ft measured in July 1957 and in Boring B-2 at elev. 227.0 ft measured on October 8, 1956. No ground water measurement is shown for Boring B-3. Data from the California Department of Water Resources database of area wells shows the water table at depths greater than 100 ft in the general area.

BCI conservatively used a design ground water level at elev. 229.0 ft in our geotechnical analysis for this site.

Ground water levels can fluctuate due to changes in precipitation, slough levels, irrigation, pumping of wells, and other factors. Seepage from the slough may cause perched ground water conditions at and below the existing slough water levels adjacent to the slough at any time of year.

5 CORROSION EVALUATION

BCI performed corrosion testing on two samples obtained from the borings completed at this site. Table 2 presents the test results for pH, resistivity, sulfates and chlorides.

Table 2: Soil Corrosion Test Summary

Boring/Sample Number	Depth (ft)	Elevation (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
A-10-B1/6	25.0-26.5	212.0 to 210.5	1,800	6.26	33.4	5.9
A-10-B2/7b	30.5-31.0	208.5 to 208.0	1,720	6.28	39.8	3.4

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

According to Caltrans Corrosion Guidelines (Version 1.0, September 2003), the site is not considered corrosive to structural elements. Appendix B contains the soil corrosion test results.

6 SEISMIC DATA AND EVALUATION

6.1 Ground Motion

BCI used seismic design procedures outlined in Caltrans 'Geotechnical Services Design Manual' (Version 1.0, August 2009) to develop the Acceleration Response Spectrum (ARS) Curve for design of the new bridge.

Based on the 2007 Caltrans Deterministic PGA Map and 2007 Fault Database, the nearest deterministic seismic sources are the Great Valley fault 1 and San Andreas fault zone (North Coast Section), each assigned the following parameters:

Fault Parameters	Great Valley fault 1	San Andreas fault zone
Fault Identification Number (FID)	20	308
Maximum Moment Magnitude (M_{max})	6.7	7.9
Site-to-Fault (R_{RUP}) Distance (km/mi)	43.81/27.2	180.1/111.9
Style of Faulting	Reverse	Right Lateral Strike Slip
Fault Dip (degrees)	15	90
Dip Direction	West	NA

Based on our boring data, SPT N_{60} blow count values, and correlations outlined in the Caltrans "Geotechnical Services Design Manual," we assign the site a small strain shear wave velocity (V_{S30}) equal to 325 meters per second for the upper 100 feet of the soil profile. Since the site is located more than 15.5 miles from the causative faults, we did not apply an adjustment factor for near-fault effects.

We used the above information to develop deterministic response spectra for the site and compared that to the Caltrans minimum deterministic response spectrum that assumes a maximum moment magnitude 6.5, vertical strike-slip event occurring at a distance of 7.5 miles. We then compared the deterministic results with the probabilistic response spectrum based on data from the 2008 United States Geological Survey (USGS) National Seismic Hazard Map for a 5% in 50 year probability of exceedance (975 year return period). We also compared our results with response spectra based on the Caltrans ARS Online tool.

We recommend a design spectrum based on the upper envelope spectral values of the combined probabilistic and minimum deterministic response spectra across the period spectrum from 0 to 5 seconds. Based on the Design ARS Curve, we assign the site a design peak ground acceleration (PGA) of 0.23 g.

We attach supporting data for our seismic design evaluation and the design acceleration response spectra in Appendix C. We attach a Regional Fault Map as Figure 3 and the Design ARS Curve as Figure 4.

6.2 Fault Rupture

The site does not lie within or adjacent to an Alquist–Priolo Earthquake Fault Zone for fault rupture hazard, and no known active faults cross the existing bridge. We therefore consider the potential for fault rupture and ground displacement to adversely affect the proposed structure as very low to nonexistent.

6.3 Seismic Slope Stability

We consider the potential for seismic slope instability in the form of landslides and mudslides at the site to be generally low, with the possible exception of local instability along the channel banks. We consider the potential for seismically induced slides on engineered fill slopes, constructed at typical gradients of 1.5H:1V or flatter, to be very low.

6.4 Liquefaction and Seismic Settlement Potential

Liquefaction can occur when relatively loose, saturated granular soil and specific soft, saturated fine-grained soils are subject to ground shaking sufficient to increase pore pressures to trigger liquefaction. Based on the soil and ground water conditions encountered in our borings, we consider the potential for detrimental liquefaction at the site to be nonexistent for the design peak ground acceleration of 0.23g.

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. Based on the soil and ground water conditions encountered in our borings, we consider the potential for detrimental seismic settlement at the site to be nonexistent for the design peak ground acceleration of 0.23g.

7 SCOUR EVALUATION

WRECO (hydraulics consultant) completed a “Bridge Design Hydraulic Study” (dated July 2010) for the State Route 32 Widening Project. That report and topographic information provided by MTCo indicate the following scour data for the bridge widening.

Table 3: Scour Data

Bridge Support	Contraction Scour (feet)	Local Scour (feet)	Total Scour Depth (feet)	Total Scour Elevation (feet)
Abutment 1	0.32	3.31	3.63	*N/A
Pier 2	0.32	7.02	7.34	224.7
Pier 3	0.32	6.68	7.00	225.0
Pier 4	0.32	5.35	5.67	228.3
Abutment 5	0.00	0.00	0.00	N/A

* Bottom of Abutment 1 is below the scour depth.

8 AS-BUILT FOUNDATION DATA

The 1957 As-Built plans show the existing bridge supported on driven, 32-ton, step-tapered steel shell piles filled with concrete. The depth of reinforcing steel is shown at minimum 12 ft. The pile driving notes shown on the As-Built LOTB indicate up to 22 ft of variation between the maximum and minimum penetration at individual supports. Final tip elevations for the overall structure vary from as high as elev. 217 ft at the abutments to elev. 192 ft at Bent 2. The large variability appears related to the local very dense gravel/cobble layers, especially between about elev. 210-220 ft (approximately 8-18 ft below channel bottom). Only the deepest piles at Bent 2 appear to have reached the underlying, very hard layer interpreted as Tuscan formation bedrock.

9 FOUNDATION RECOMMENDATIONS

9.1 Foundation Data and Loading

The subsurface conditions encountered in our borings indicate that the site is suitable for driven concrete piles. Driven Class 90 (Alt X) precast, prestressed concrete piles were selected for abutment support. Driven 15-inch precast, prestressed concrete pile extensions (Slab Bridge Pile Details, Section B-B, Sheet xs1-230, 4/4/1997) were selected for pier support. Drilling to assist pile driving may be required for driven piling to penetrate locally dense soil layers and achieve specified tip elevations.

Due to locally coarse sediments and shallow (perched) ground water, cast-in-drilled-hole (CIDH) piles would require special installation measures, including casing, slurry drilling methods and the use of minimum 24-inch diameter CIDH piles for tremie concrete placement. Therefore, we do not recommend the use of CIDH piles.

We do not recommend the use of spread footings due to the weak, near-surface soils at the abutments and potential channel scour.

MTCo provided the following foundation design information in Tables 4 and 5.

Table 4: Foundation Design Data Provided By MTCo

Foundation Design Data									
Support No.	Design Method (WSD or LRFD)	Pile Type	Finish Grade Elev. (ft)	Original Ground Elev. (ft)*	Pile Cut-off Elevation (ft)*	Pile Cap Size (ft)		Permissible Settlement – Service Load (in)	Number of Piles per Support
						B	L		
Abut 1	WSD	Class 90 (Alt X)	239.1	N/A	236.3	3.5	68.4	1	8
Pier 2	LRFD	Class 90 (T=15)	232.0	232.0 (No Scour) 224.7 (Scour)	241.1	NA	NA	1	10
Pier 3	LRFD	Class 90 (T=15)	232.0	232.0 (No Scour) 225.0 (Scour)	241.2	NA	NA	1	10
Pier 4	LRFD	Class 90 (T=15)	234.0	234.0 (No Scour) 228.3 (Scour)	241.3	NA	NA	1	10
Abut 5	WSD	Class 90 (Alt X)	239.3	N/A	236.5	3.5	68.4	1	8

Table 5: Foundation Design Loads Provided By MTCo

Foundation Design Loads												
Support No.		**Service-I Limit State (kips)**			**Strength Limit State (Controlling Group, kips)**				**Extreme Limit State (Controlling Group, kips)**			
		Total Load		Permanent Loads Per Support	Compression		Tension		Compression		Tension	
		Per Support	Per Pile		Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	-	610	75	290	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	No Scour	870	90	520	1260	130	0	0	520	55	0	0
	Scour	960	100	610	1370	140						
Pier 3	No Scour	770	80	450	1120	115	0	0	450	45	0	0
	Scour	860	90	540	1220	125						
Pier 4	No Scour	850	85	500	1230	125	0	0	500	55	0	0
	Scour	920	95	570	1320	135						
Abut 5	-	610	75	290	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

9.2 Foundation Recommendations and Pile Data Table

BCI used the above foundation design data and loading conditions to evaluate bent foundations using AASHTO LRFD Bridge Design Specifications-4th Edition with current Caltrans Amendments. We evaluated abutment foundations using Caltrans November 2003 Bridge Design Specifications for foundations using Working Stress Design methods. We present our foundation recommendations in Tables 6, 7 and 8.

Table 6: Foundation Recommendations for Abutments

Abutment Foundation Design Recommendations										
Support	Pile Type	Cut-off Elev. (ft.)	**LRFD Service-I Limit State Load – Compression (kips)**			Required Nominal Resistance (kips)		Design Tip Elevations (ft.)	Specified Tip Elevation (ft.)	Nominal Driving Resistance (kips)
			Per Support		Per Pile	Comp.	Tens.			
			Total	Permanent						
Abut 1	Class 90 (Alt. X)	236.3	610	290	75	150	0	200.0(a)	200.0	150
Abut 5	Class 90 (Alt. X)	236.5	610	290	75	150	0	200.0(a)	200.0	150

Notes: 1) Design tip elevations for **Abutments** are controlled by (a) Compression.

Table 7: Foundation Recommendations for Piers

Bent Foundation Design Recommendations											
Support	Pile Type	Cut-off Elev. (ft.)	LRFD Service-I Limit State Load Per Support – Compression (kips)	Total Permissible Support Settlement (in.)	Required Factored Nominal Resistance (kips) Per Pile				Design Tip Elevations (ft.)	Specified Tip Elevations (ft.)	Nominal Driving Resistance Required (kips)
					Strength Limit		Extreme Event				
					Comp $\varphi = 0.7$	Tens. $\varphi = 0.7$	Comp $\varphi = 1.0$	Tens $\varphi = 1.0$			
Pier 2	15" PC/PS Pile Extensions	241.1	610	1.0	140	0	55	0	188.0 (a),(b)	188.0	200
Pier 3	15" PC/PS Pile Extensions	241.2	540	1.0	125	0	45	0	188.0 (a),(b)	188.0	200
Pier 4	15" PC/PS Pile Extensions	241.3	570	1.0	135	0	55	0	188.0 (a),(b)	188.0	200

Notes:
1) Design tip elevations for **Piers** are controlled by (a) Compression (Strength Limit), (b) Scour, respectively.
2) The nominal driving resistance is equal to the required nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the required nominal resistance due to scour.

Based on our analysis presented in the following sections, BCI presents our recommended Pile Data Table as Table 8:

Table 8: Pile Data Table

Pile Data Table						
Support	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft.)	Specified Tip Elevation (ft.)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 90 (Alt. X)	150	0	200.0(a)	200.0	150
Pier 2	15" PC/PS Pile Extensions	200	0	188.0(a)(b), 210.0 (c)	188.0	200
Pier 3	15" PC/PS Pile Extensions	180	0	188.0(a)(b), 210.0 (c)	188.0	200
Pier 4	15" PC/PS Pile Extensions	200	0	188.0(a)(b), 210.0 (c)	188.0	200
Abut 5	Class 90 (Alt. X)	150	0	200.0(a)	200.0	150

Notes:

1) Design tip elevations for **Abutments** are controlled by (a) Compression.
2) Design tip elevations for **Piers** are controlled by (a) Compression (Strength Limit), (b) Scour, and (c) Lateral, respectively. The Design Tip Elevations for lateral loading were determined by MTCo.
3) The nominal driving resistance is equal to the required nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the required nominal resistance due to scour.

9.3 ENGNEERING PARAMETERS

BCI developed generalized engineering parameters for this project based on the following:

- Average unit weight values based on our laboratory tests, local experience and published typical values.
- Average cohesion values based on unconfined compressive strength testing, field pocket penetrometer testing, and published blow count correlations.
- Friction angles based on published blow count correlations.
- Modulus and E_{50} strain values for lateral pile analysis obtained from the July 2004 LPILE Plus 5.0 Technical Manual for appropriate soil type and consistency.
- Engineering experience and judgment.
- BCI used a ground water elevation of 229.0 ft for design.

9.3.1 Compressive Resistance

We used the generalized soil parameters in Table 9 in our bearing capacity analysis.

Table 9: Generalized Soil Parameters

Elevation	Soil Type	Total Unit Weight (pcf)	Buoyant Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
239.0 to 223.5	Silt / Clay / Sandy Clay	119	57	---	1,400
223.5 to 217.5	Sand	106	44	27	---
217.5 to 213.0	Clayey Sand	107	45	36	---
213.0 to 191.0	Silt / Clay / Sandy Clay	106	44	---	1,400
191.0 to 157.0	Gravel and Sedimentary Rock	107	45	38	---

9.3.2 LPILE Parameters

MTCo requested that BCI provide LPILE parameters for use in their equivalent column length analysis. Table 10 provides our recommended LPILE parameters for the abutment and bent piles.

Table 10: LPILE Parameters

Elevation	L-Pile Soil Type (p-y curve model)	Unit Weight (pci)	Friction Angle (degrees)	Cohesion (psi)	ε_{50} (dim.)	Modulus, k (pci)
239.0 to 229.0	Stiff Clay w/o Free Water (Reese)	0.0689	--	10.4	0.007	**
229.0 to 223.5	Stiff Clay with Free Water (Reese)	0.0328 (submerged)	--	10.4	0.007	500
223.5 to 217.5	Sand (Reese)	0.0252 (submerged)	27	--	--	20
217.5 to 213.0	Sand (Reese)	0.0619 (submerged)	36	--	--	125
213.0 to 191.0	Stiff Clay with Free Water (Reese)	0.0250 (submerged)	--	13.9	0.007	600
191.0 to 157.0	Sand (Reese)	0.0258 (submerged)	38	--	--	125

** L-pile program internally calculates k value for clay without Free Water (Reese).
Neglect lateral resistance at the piers above scour elevation.

9.4 Abutment Piles (Class 90)

In accordance with current Caltrans specifications, we used the Working Stress Design (WSD) for the abutment piles. BCI evaluated Alternative "X" Class 90 piles with a T dimension of 12 inches for the abutments. BCI presents the results of our compressive resistance and settlement analysis below. No tension demand is indicated for abutment piles.

9.4.1 Compressive Resistance

The tips of the Class 90 precast, prestressed concrete (PPC) piles will bear in very stiff silt and sandy clay and medium dense clayey sand about 32 ft below the existing channel bottom elevation.

Our calculations indicate that the nominal compressive resistance of the PPC piles can be obtained through about 10% end bearing and 90% skin friction. Actual contributions to end bearing and skin friction could vary depending on how the load is transferred to the piles. We neglected the approach fill in our skin friction and end bearing analysis. We modeled the top of abutment piles at elev. 236.3 ft for both abutments.

We determined the compressive resistance using the Federal Highway Administration's Driven 1.2 (March 20, 2001) computer program developed by Blue-Six Software, Inc.

Refer to the Driven output files in Appendix D for the analysis results.

9.4.2 Settlement

We calculated an immediate settlement of less than 0.5 inch for the Service-I Limit State total load (per pile) using the method outlined in Section 16-10 of Foundation Analysis and Design, 5th edition, Joseph E. Bowles, 1996. We include the pile settlement calculations in Appendix D.

Our calculated pile settlement is less than the permissible settlement of 1-inch specified for the structure foundations. We do not anticipate significant long-term settlement due to the competent soil conditions at and below the specified tip elevations.

9.4.3 Lateral Load Analysis

MTCo indicates that pile compression loading will control at abutments and did not request pile lateral load analysis for design of the abutment piles. BCI provides LPILE parameters in Section 9.3.2 for use in lateral pile analysis if desired.

9.4.4 Negative Skin Friction

We do not anticipate negative skin friction at the abutments given the competent soil conditions and nominal new embankment heights.

9.5 Pier Piles

We used AASHTO LRFD Bridge Design Specifications-4th Edition and current Caltrans Amendments for evaluating driven 15-inch precast, prestressed concrete pile extensions (Caltrans Slab Bridge Details, Sheet xs1-23) at the piers. BCI presents the results of our compressive resistance and settlement analysis below. No tension demand is indicated for pier piles.

9.5.1 Compressive Resistance

The tips of the 15-inch precast, prestressed concrete pile extensions will bear in very dense gravel and/or rock about 44 ft below the existing channel bottom elevation. Our calculations indicate that the nominal compressive resistance of the piles will essentially be obtained through end bearing. Therefore, we conservatively base our pile tip elevations on end bearing contribution only.

We determined the required nominal compressive resistance using the Federal Highway Administration's Driven 1.2 (March 20, 2001) computer program developed by Blue-Six Software, Inc.

The Driven 1.2 computer program calculates compressive resistance for piles with square sides (pile width input, not diameter). For pier piles, we considered an equivalent pile width of 11.8 inches for the pile perimeter (skin friction), and 13.3 inches for the pile tip area (end bearing). We then used an equivalent pile width of 13.3 inches for end bearing contributions and reduced the skin friction contributions by a factor of 0.89 (i.e., 11.8/13.3 = 0.89). We modeled the top of pier piles at elev. 232.0 ft with estimated total scour at elev. 224.7 ft (lowest elevation).

BCI determined the required factored nominal resistance by comparing the highest Factored Strength Limit Load (Geotechnical Resistance Factor = 0.7) with the highest Extreme Event Load (Resistance Factor = 1.0). We then used the higher value as the required factored nominal resistance. In this case, the Factored Strength Limit Load (140 kips/0.7 = 200 kips per pile) is controlling over the 55 kips per pile for the Extreme Event. We conservatively used a Factored Strength Limit Load of 200 kips per pile to estimate the design tip elevation for all pier piles using the Driven software.

Refer to the Driven output files in Appendix E for additional information.

9.5.2 Settlement

We calculated an immediate settlement of less than 0.5 inch for the Service-I Limit State total load (per pile) using the method outlined in Section 16-10 of Foundation Analysis and Design, 5th edition, Joseph E. Bowles, 1996. We include the pile settlement calculations in Appendix E.

Our calculated pile settlement is at the 1-inch permissible settlement level specified for the structure foundations. We do not anticipate significant long-term settlement due to the competent soil conditions at and below the equivalent footing level.

9.5.3 Lateral Load Analysis

BCI provides LPILE parameters in Section 9.3.2 for MTCo to use in their equivalent column length and lateral pile analysis for the piers.

9.5.4 Negative Skin Friction

We do not anticipate negative skin friction at the piers.

10 APPROACH FILLS

10.1 Fill Materials

Embankments will be constructed using imported borrow material, supplemented with material excavated from shallow on-site cuts and existing approach embankment fill. The source(s) 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.

Expansive soil (Expansion Index > 50 and Sand Equivalent < 20) should not be used as fill within 5 ft of the abutment backwall.

10.2 Slope Geometry and Stability

The existing approach fill and underlying native soil are competent for support of the planned embankments.

10.3 Settlement

We estimate nominal (< 1-inch) settlement for fill loads up to 5 ft high. No waiting period is necessary prior to construction of bridge abutment foundations.

10.4 Lateral Earth Pressures

The following equivalent fluid weights (EFWs) may be used to design the abutment walls and wing walls for Abutments 1 and 4 assuming level backfill conditions:

Condition	EFW Static	EFW Seismic
Active	36 lb/ft^3	42 lb/ft^3
At-Rest	56 lb/ft^3	66 lb/ft^3
Passive	220 lb/ft^3	205 lb/ft^3

The EFWs shown above assume embankment fill meeting the requirements of Caltrans standard for Structure Backfill, a soil unit weight of approximately 125 pcf, a minimum angle of internal friction equal to 34 degrees, and that drainage is placed behind the walls 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.

We 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 0.12 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. We calculated the above static EFWs using methods presented in the 1982 Naval Facilities (NAVFAC) Design Manual 7.2.

Apply the resultant of the seismic active and at-rest pressures at a depth 0.5H from the base of the wall, where H equals the wall height in feet. The passive pressures are applicable for concrete placed directly against undisturbed soil or compacted fill.

As noted in the Caltrans Seismic Design Criteria (SDC), the maximum passive pressure is 5.0 ksf for longitudinal abutment response, which must be used with the proportionality factor presented in Section 7.8.1 of the SDC.

For surcharge loads, apply an additional uniform lateral load behind the wall equivalent to (0.30)x(surcharge pressure).

Use a coefficient of friction of 0.45 to resist sliding for concrete placed on native undisturbed soil or compacted fill.

11 CONSTRUCTION CONSIDERATIONS

Where referenced below, “Standard Specifications” refers to Caltrans Standard Specifications (May 2006).

11.1 Embankment Waiting Period and Settlement Monitoring

No waiting period is required prior to construction of bridge abutment piling.

11.2 Abutment and Pier Piles

Piles shall conform to Section 49-1 of the Standard Specifications.

At the abutments, perform predrilling through embankment when the depth of new embankment is 5 feet or greater in accordance with Section 49-1.06 of the Standard Specifications.

Difficult pile installation is anticipated due to the presence of locally dense sand and gravel layers above the specified tip elevations. Drilling to assist pile driving may be necessary to achieve the specified tip elevations. Drilling should be performed in accordance with Section 49-1.05 of the Standard Specifications, *except the drill hole should be no greater than 8-inches in diameter at the abutments and no greater than 10-inches at the piers. Do not drill below elev. 213 ft.*

Jetting or vibratory hammers should not be used to obtain the specified pile penetration.

Verify pile capacity during driving using energy equations in accordance with Caltrans Standard Specification 49-1.08 (Modified Gates Formula).

Piles that achieve the required nominal driving resistance (i.e., 150 kips at abutments and 200 kips at piers) during initial driving at/within five feet above specified tip elevation are acceptable. Otherwise, contact BCI to review pile driving records and construction conditions for additional pile acceptance criteria if needed.

Pile driving should not negatively impact the existing bridge structures since they are supported on piles.

11.3 Temporary Shoring

The contractor is responsible for design and construction of excavation sloping and shoring in accordance with CalOSHA Standards.

11.4 Dewatering

Excavations extending below the slough water level will require dewatering and/or diking/diversion methods to construct abutment foundations in the “dry.”

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, BCI should be retained to:

1. Review and provide written comments on the (civil, structural) plans and specifications prior to construction.
2. Monitor construction to check and document our report assumptions. At a minimum, we should monitor pile installation.
3. Update this report if:
    - design changes occur
    - 2 years or more lapse between this report and construction
    - 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 addenda, letters, and discussions.

13 LIMITATIONS

This report should only be used for design and construction of the Dead Horse Slough Bridge (Widen) project as described herein. Do not use or rely upon this report for different locations or improvements without the written consent of BCI

BCI performed services in accordance with the generally accepted geotechnical standard of practice currently used in this area. Where referenced, we used ASTM and Caltrans Standards as a general (not strict) *guideline* only. We do not warranty our services.

BCI based this report on the current site and project conditions. We assumed the soil, rock, and ground water conditions encountered in our exploratory borings are representative of the subsurface conditions across the site. Actual conditions between borings could be different. Ground water may be higher in other locations than measured in the borings.

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

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

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.

APPENDIX A

Figure 1 – Vicinity Map

Figure 2 – Geologic Map

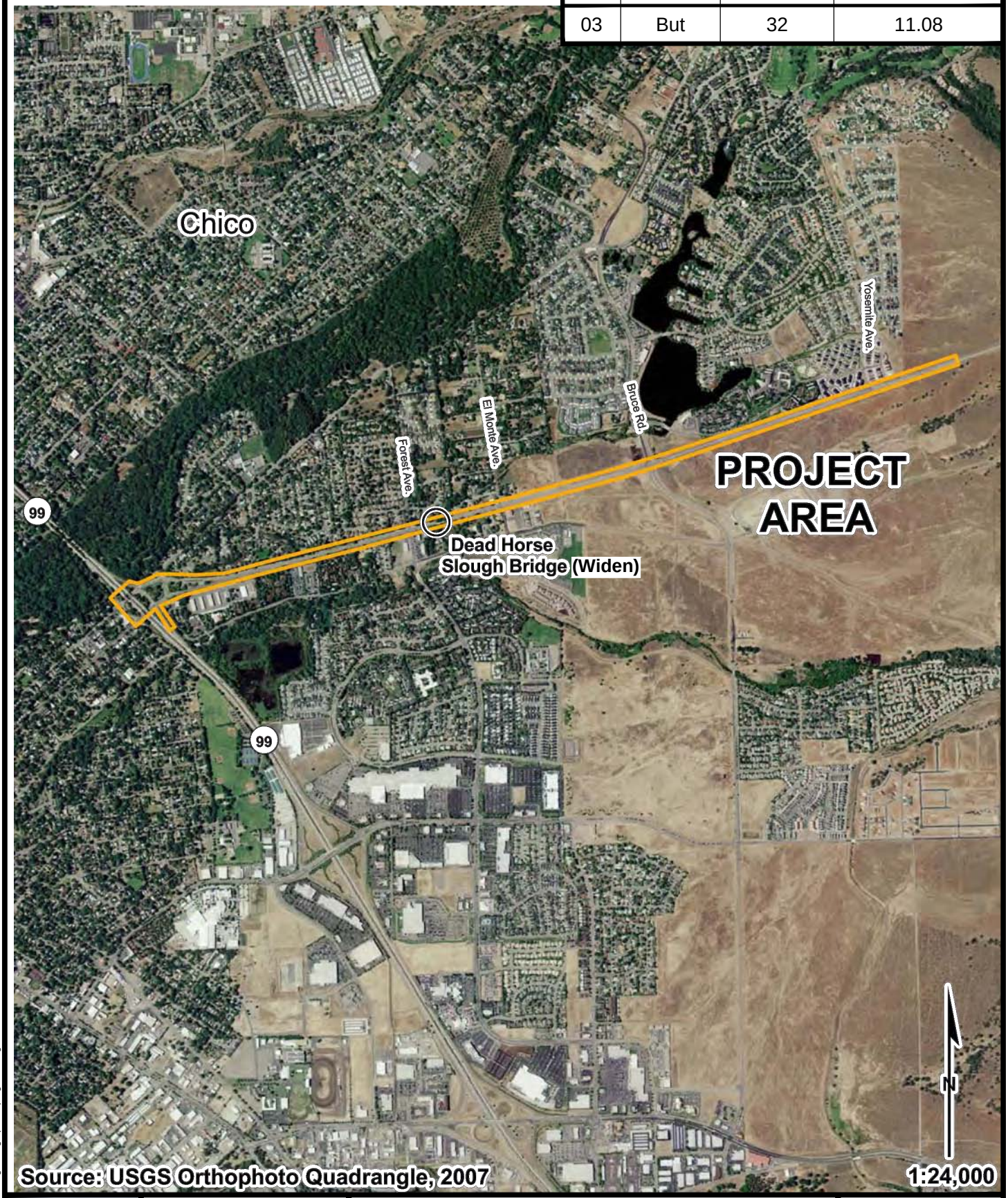
Figure 3 – Regional Fault Map

Figure 4 – Design ARS Curve

Log of Test Borings Sheets 1-5

Log of Test Borings Sheet Checklist

DIST	COUNTY	ROUTE	POST MILES
03	But	32	11.08



blackburn consulting

2491 Boatman Avenue
West Sacramento,CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

VICINITY MAP

Dead Horse Slough Bridge (Widen)
EA 1E4901 / CU 03
Chico, California

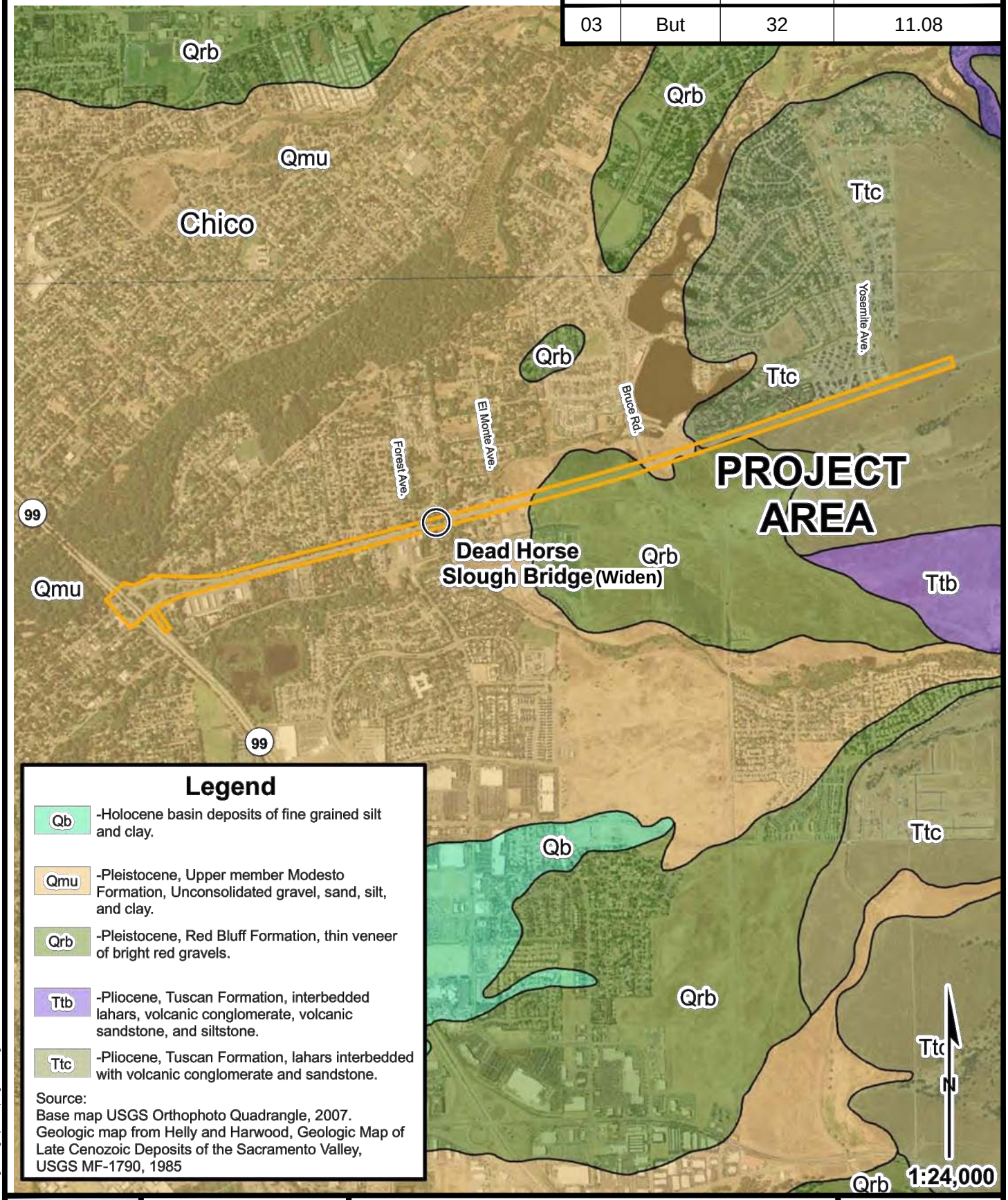
File No. 1202.1

September 2010

Figure 1

9/20/2010 1202.1 Dead Horse Slough Bridge (Widen) Figure 1.dwg

DIST	COUNTY	ROUTE	POST MILES
03	But	32	11.08



Legend

- Qb -Holocene basin deposits of fine grained silt and clay.
- Qmu -Pleistocene, Upper member Modesto Formation, Unconsolidated gravel, sand, silt, and clay.
- Qrb -Pleistocene, Red Bluff Formation, thin veneer of bright red gravels.
- Ttb -Pliocene, Tuscan Formation, interbedded lahars, volcanic conglomerate, volcanic sandstone, and siltstone.
- Ttc -Pliocene, Tuscan Formation, lahars interbedded with volcanic conglomerate and sandstone.

Source:
Base map USGS Orthophoto Quadrangle, 2007.
Geologic map from Helly and Harwood, Geologic Map of Late Cenozoic Deposits of the Sacramento Valley, USGS MF-1790, 1985

blackburn consulting
2491 Boatman Avenue
West Sacramento,CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
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GEOLOGIC MAP

Dead Horse Slough Bridge (Widen)
EA 1E4901 / CU 03
Chico, California

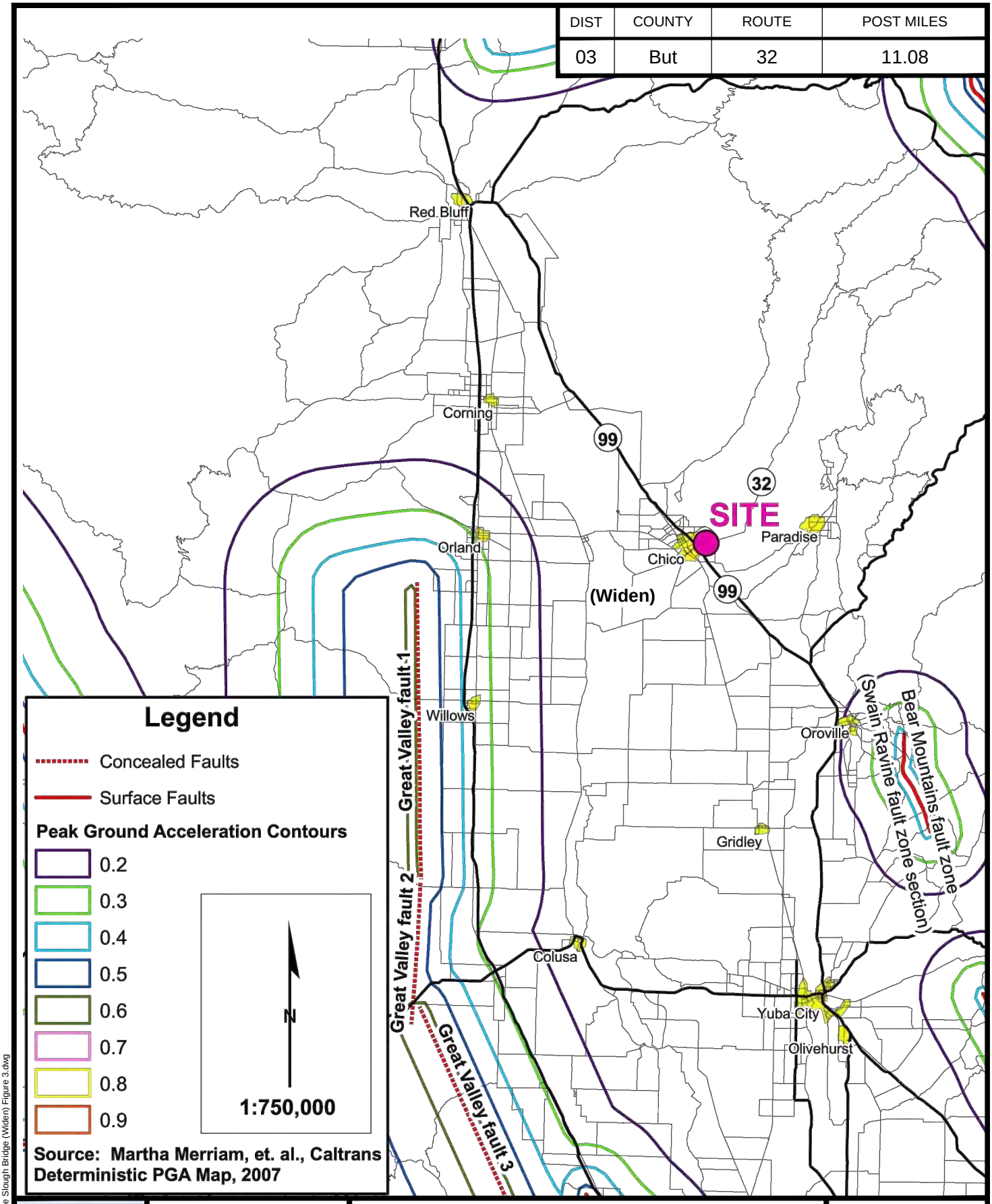
File No. 1202.1

September 2010

Figure 2

9/20/2010 1202.1 Dead Horse Slough Bridge (Widen) Figure 2.dwg

DIST	COUNTY	ROUTE	POST MILES
03	But	32	11.08



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2491 Boatman Avenue
West Sacramento,CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
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REGIONAL FAULT MAP

Dead Horse Slough Bridge (Widen)
EA 1E4901 / CU 03
Chico, California

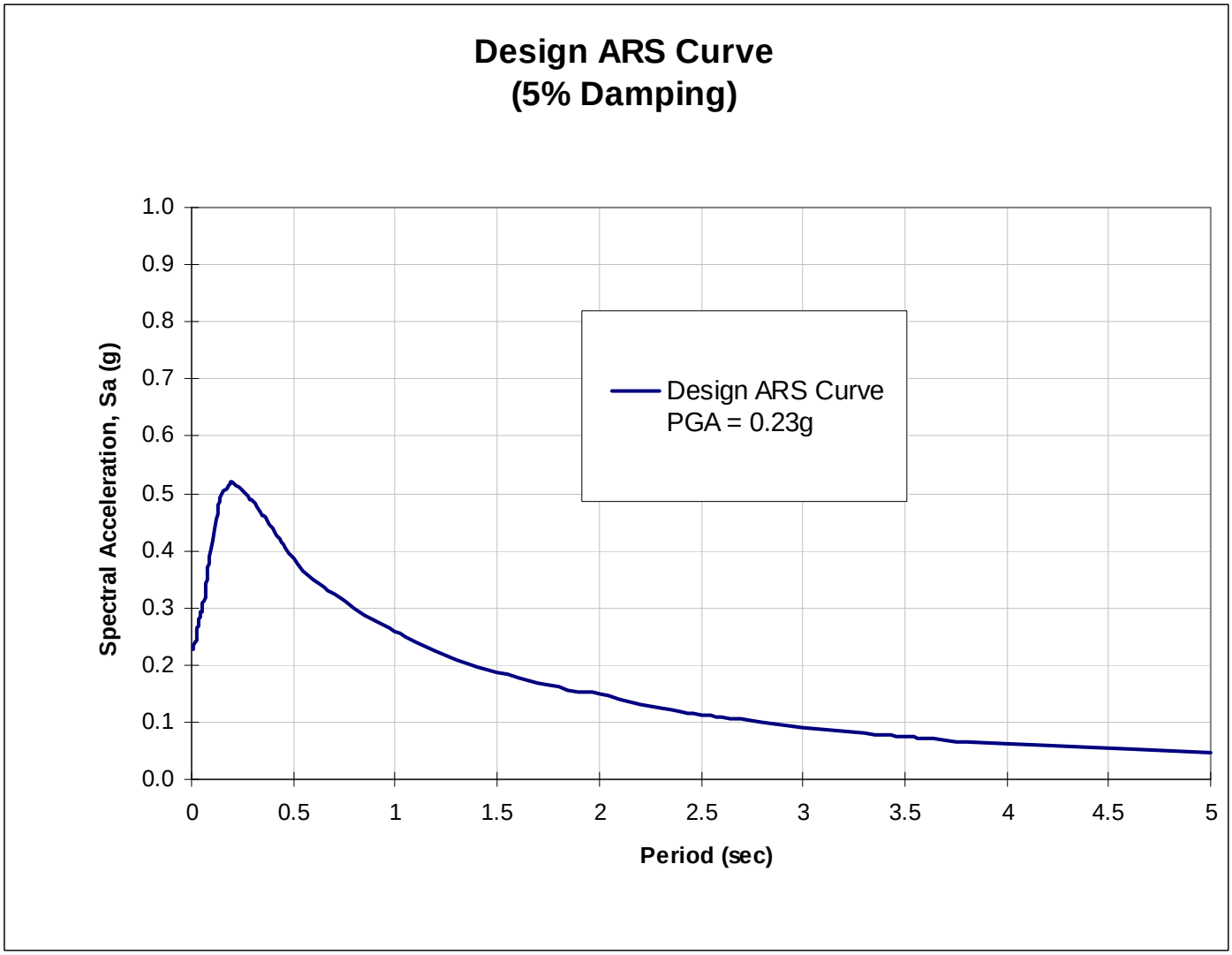
File No. 1202.1

September 2010

Figure 3

9/20/2010 1202.1 Dead Horse Slough Bridge (Widen) Figure 3.dwg

DIST	COUNTY	ROUTE	POST MILES
03	But	32	11.08



Reference: Geotechnical Services Design Manual (Version 1.0, August 2009) and Caltrans Seismic Design Criteria, Appendix B, Revised 9/11/09.

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2491 Boatman Avenue
West Sacramento, CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

DESIGN ARS CURVE

Dead Horse Slough Bridge (Widen)
EA 1E4901 / CU 03
Chico, California

File No. 1202.1

September 2010

Figure 4

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	But	32	10.1/12.7	186	190

REGISTERED ENGINEERING GEOLOGIST — DATE 9/22/10

PLANS APPROVAL DATE

PROFESSIONAL GEOLOGIST — William E. Nichols No. 2229 Exp. 1-31-12 — CERTIFIED ENGINEERING GEOLOGIST — STATE OF CALIFORNIA

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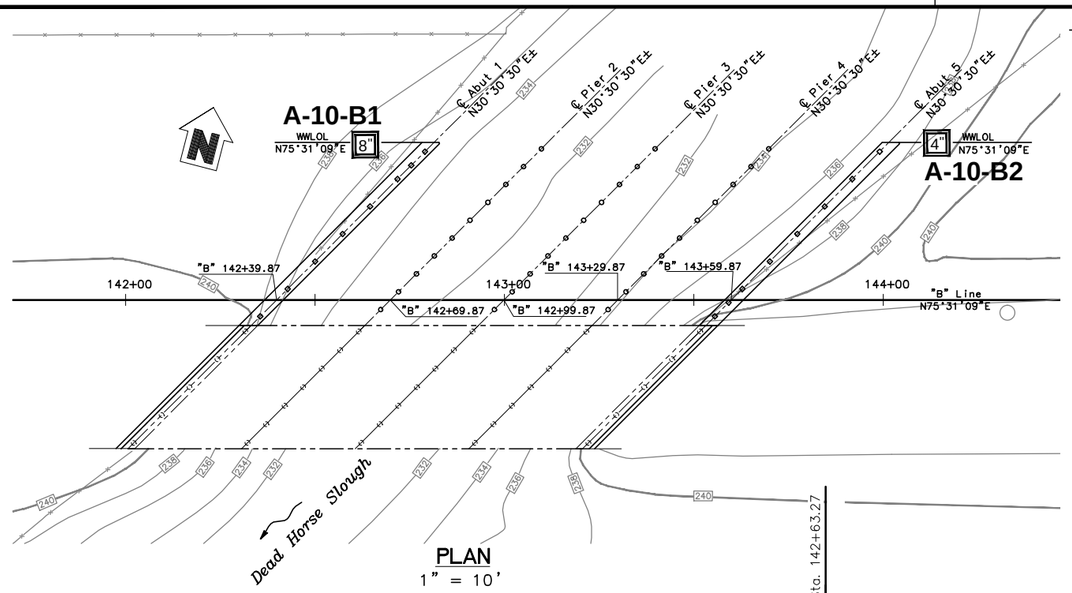
2491 BOATMAN AVENUE
WEST SACRAMENTO, CA 95691

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

CAPITAL PROJECT SERVICES
411 MAIN STREET
CHICO, CA 95927



PLAN
1" = 10'

LEGEND:

- ○ Indicates Driven Pile
- ◌ Indicates Existing Pile
- ——— Indicates Existing Structure

NOTES:

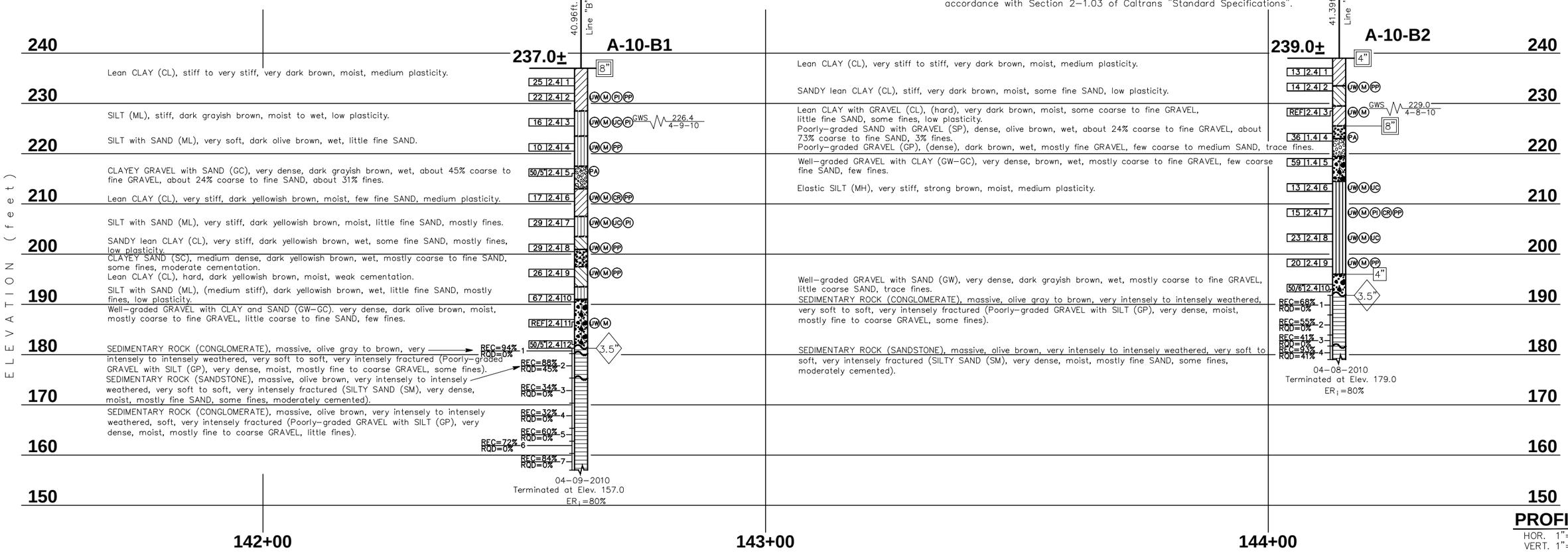
1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2007). See Log of Test Borings No. 2 and 3, "Soil Legend" and Log of Test Borings No. 4 "Rock Legend".
2. Standard Penetration tests were performed in accordance with 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. 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.
5. The length of each sampled interval is shown graphically on the boring log.
6. Consistency of soils shown in ( ) where estimated.
7. Ground water surface (GWS) reflects the fluid level in the boring on the specified date. Ground water surface is subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
8. Electronic media for plan view provided by Mark Thomas & Company, Inc., 7-26-2010.
9. Boring elevations are approximate and based on "Foundation Plan", received 7-26-2010.
10. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".

BENCHMARK

P1434 (PID: KS1933) NGS cap on rod in monument well at east 8th & Linden. Elev 210.91

TBM #34 Set mag nail in AC at northerly shoulder SR 32. Elev 242.16

TBM #31 Set mag nail in AC at northerly shoulder SR 32. Elev 246.28



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

65% SUBMITTAL

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	J. PASSALACQUA PROJECT ENGINEER	BRIDGE NO. 12-0135	DEAD HORSE SLOUGH BRIDGE (WIDEN)
FUNCTIONAL SUPERVISOR X	DRAWN BY: M. ROBERTSON	FIELD INVESTIGATION BY: K. CHAPMAN			POST MILES 11.08	LOG OF TEST BORINGS 1 OF 5
	CHECKED BY: W. E. NICHOLS	BCI File No. 1202.1				

FOUNDATION PLAN SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 03
EA 1E4901

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 13 OF 17

FILE => $REQUEST

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

9/22/2010 1202.1 Dead Horse Slough Bridge (Widen) LOTB.dwg

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	But	32	10.1/12.7	187	190

REGISTERED ENGINEERING GEOLOGIST — DATE 9/22/10

PLANS APPROVAL DATE

PROFESSIONAL GEOLOGIST William E. Nichols No. 2229 Exp. 1-31-12 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

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7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

CAPITAL PROJECT SERVICES
411 MAIN STREET
CHICO, CA 95927

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

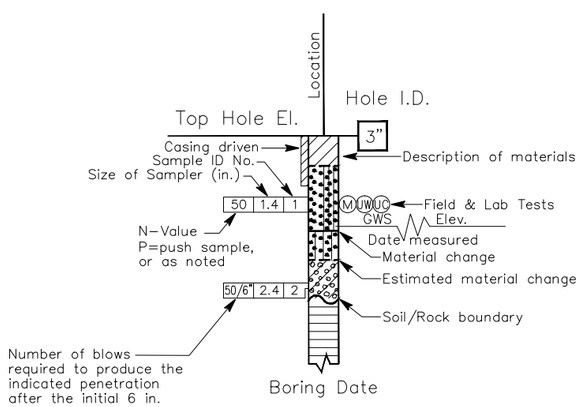
BOREHOLE IDENTIFICATION

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

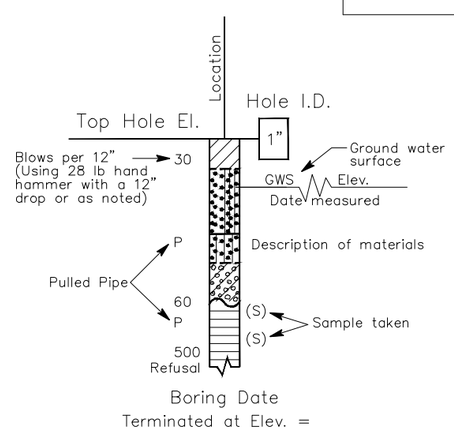
NOTE: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

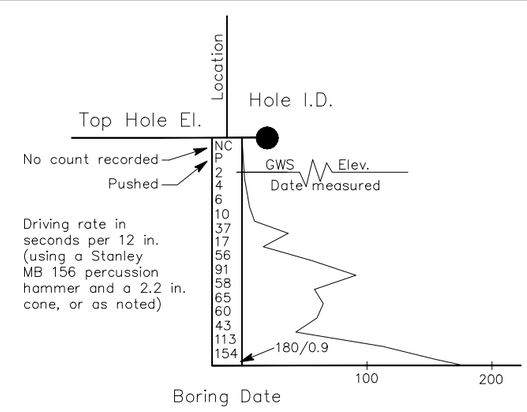
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be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



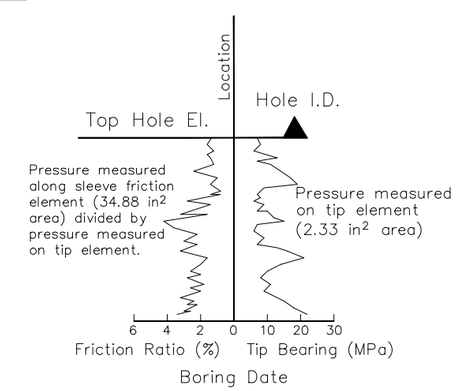
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

65% SUBMITTAL

SOIL LEGEND

DEAD HORSE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 2 OF 5

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	J. PASSALACQUA PROJECT ENGINEER	BRIDGE NO. 12-0135	SHEET 14 OF 17
FUNCTIONAL SUPERVISOR X	DRAWN BY: M. ROBERTSON CHECKED BY: W. E. NICHOLS	FIELD INVESTIGATION BY: K. CHAPMAN BCI File No. 1202.1			POST MILES 11.08	

FOUNDATION PLAN SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 03
EA 1E4901

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

FILE => $REQUEST

9/22/2010 1202.1 Dead Horse Slough Bridge (Widen) LOTB.dwg

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

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	But	32	10.1/12.7	188	190

REGISTERED ENGINEERING GEOLOGIST — DATE 9/22/10

PROFESSIONAL GEOLOGIST William E. Nichols No. 2229 Exp. 1-31-12 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

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SACRAMENTO, CA 95826

CAPITAL PROJECT SERVICES
411 MAIN STREET
CHICO, CA 95927

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

GROUP SYMBOLS AND NAMES

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

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

65% SUBMITTAL

SOIL LEGEND

DEAD HORSE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 3 OF 5

ENGINEERING SERVICES

FUNCTIONAL SUPERVISOR: X

DRAWN BY: M. ROBERTSON

CHECKED BY: W. E. NICHOLS

GEOTECHNICAL SERVICES

FIELD INVESTIGATION BY: K. CHAPMAN

BCI File No. 1202.1

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

J. PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 12-0135
POST MILES 11.08

CU 03
EA 1E4901

FOUNDATION PLAN SHEET (ENGLISH) (REV. 06-01-09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 15 OF 17

FILE => $REQUEST

9/22/2010 1202.1 Dead Horse Slough Bridge (Widen) LOTB.dwg

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

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

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
03	But	32	10.1/12.7	189	190

REGISTERED ENGINEERING GEOLOGIST — DATE 9/22/10

PROFESSIONAL GEOLOGIST William E. Nichols No. 2229 Exp. 1-31-12 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

PLANS APPROVAL DATE

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

BLACKBURN CONSULTING
2491 BOATMAN AVENUE
WEST SACRAMENTO, CA 95691

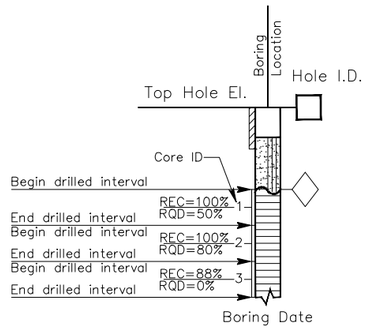
MARK THOMAS & CO., INC.
7300 FOLSOM BLVD STE 203
SACRAMENTO, CA 95826

CAPITAL PROJECT SERVICES
411 MAIN STREET
CHICO, CA 95927

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 hairline fractures, or veinlets. Rock is significantly weakened.
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe–Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated).	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete.		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

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

65% SUBMITTAL

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

ROCK LEGEND

DEAD HORSE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 4 OF 5

ENGINEERING SERVICES

FUNCTIONAL SUPERVISOR: X

DRAWN BY: M. ROBERTSON

CHECKED BY: W. E. NICHOLS

GEOTECHNICAL SERVICES

FIELD INVESTIGATION BY: K. CHAPMAN

BCI File No. 1202.1

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

J. PASSALACQUA
PROJECT ENGINEER

BRIDGE NO. 12–0135
POST MILES 11.08

FOUNDATION PLAN SHEET (ENGLISH) (REV. 06–01–09)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 03
EA 1E4901

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 16 OF 17

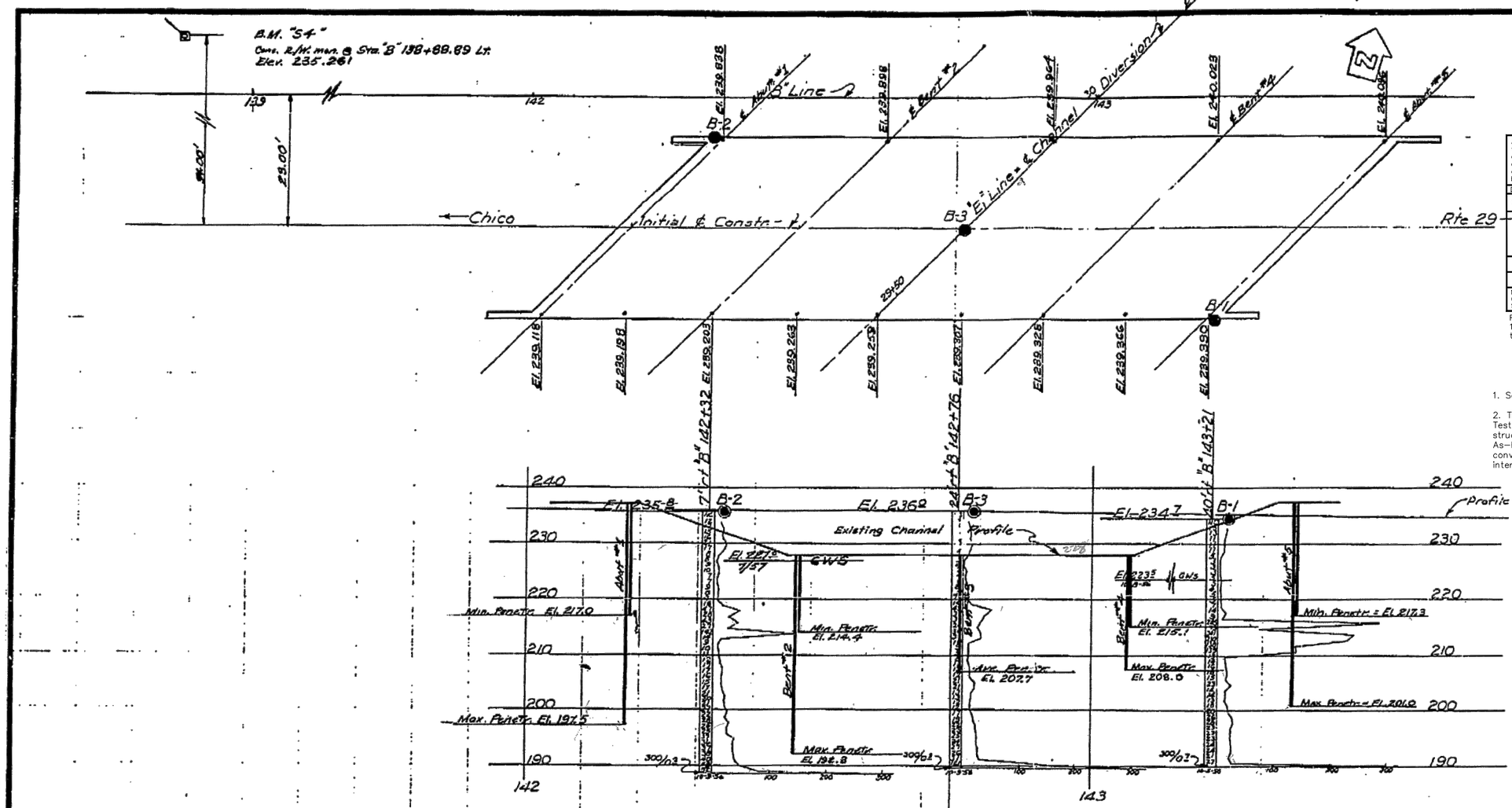
FILE => $REQUEST

9/22/2010 1202.1 Dead Horse Slough Bridge (Widen) LOTB.dwg

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEARS	SHEET NO.	TOTAL SHEETS
7	CAL.	S-7601		34	39

DIST.	COUNTY	ROUTE	SECTION	SHEET NO.	TOTAL SHEETS
III	But.	32	B	5	5

February 25, 1957



BM B-139

Spike in crack of NE conc. Wing Wall at Bridge on Forest Ave. 200'± lt "B" 138+60 Elev. 235.56

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

DIST.	COUNTY	ROUTE	POST MILES–TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	But	32	10.1/12.7	190	190

W. Eric Nichols 9/22/10
REGISTERED ENGINEERING GEOLOGIST DATE

DEAD HORSE SLOUGH BRIDGE (WIDEN)

LOG OF TEST BORINGS 5 of 5

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

CU: 03
EA: 1E4901

BRIDGE NO. 12–0135

SHEET	OF
17	17

Revisions made to this Log of Test Borings from the original 1963 Log of Test Borings are the addition of the following table and notes.

Boring	Station	Offset from Line "B"
B–1	143+44.1	16.3 ft Rt
B–2	142+55.3	16.3 ft Lt
B–3	142+99.7	00.3 ft Rt

1. See Log of Test Borings 1 of 5 for stationing.

2. The table above are the locations for the As-Built Test Borings referenced to the proposed new structure location. This table is presented on the As-Built Log of Test Boring sheet for the convenience of any bidder, contractor or other interested party.

PROFESSIONAL GEOLOGIST
William E. Nichols
No. 2229
Exp. 1-31-12
ENGINEERING GEOLOGY
STATE OF CALIFORNIA

AS BUILT PLANS
CONTRACT 57-3TC37-F
M. E. Tyson Sr. 8 NOV. 1957.
R.E.

Chico to Hog Springs
III-But-47-B
Contract 57-3TC37-F
S-701(1)

BRIDGE DEPARTMENT

FIELD STUDY	
	by: R. Hoover 10-56
DRAWN	by: J. Barr 10-56
CHECKED	by: C. Hurst 12-56

Approval Recommended by: [illegible]
Date: [illegible]

Classification of Material Based on Standard Grade Size Limits

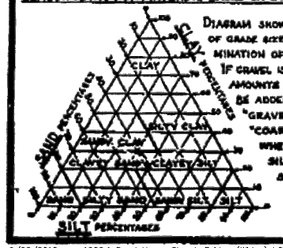


Diagram showing the basis for estimates of grade size distribution used in determination of class names.

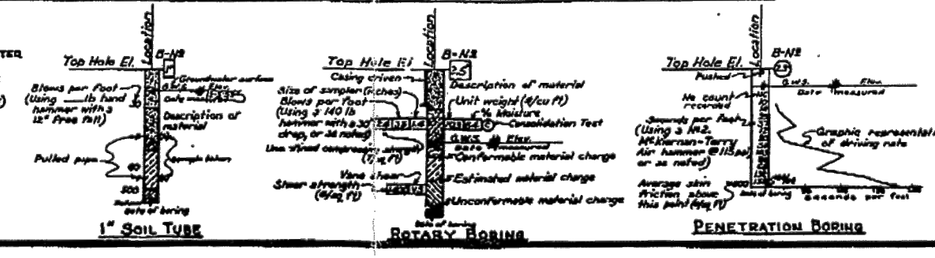
If gravel is present in appreciable amounts the term "gravelly" may be added to the class name, viz. "gravelly sand." The terms "coarse", "medium" and "fine" when used to describe sand, silt and gravel, refer to standard grade size limits.

Legend of Earth Materials

- Gravel
- Sand
- Silt
- Clay
- Sandy Clay or Clayey Sand
- Sandy Silt or Silty Sand
- Silty Clay or Clayey Silt
- Peat and/or Organic Matter
- Fill Material
- Igneous Rock
- Sedimentary Rock
- Metamorphic Rock

Legend of Boring Operations

- Plan of any boring
- Penetrometer
- 2¼" Cone Penetrometer
- Sampler boring (dry)
- Rotary boring (wet)
- Auger boring (dry)
- Jet boring
- Core boring
- Test pit



Notes

The contractor's attention is directed to Section 2, Article (c) of the Standard Specifications and to the Special Provisions accompanying this set of plans.

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

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

DEAD HORSE SLOUGH

LOG OF TEST BORINGS

SCALE 1" = 10'	BRIDGE 12-135	FILE E-72	DRAWING E-5205-5

34

9/22/2010 1202.1 Dead Horse Slough Bridge (Widen) LOTB.dwg

Log of Test Boring (LOTB) Sheet Checklist

This checklist shall be used by the *checker* in his/her evaluation of a LOTB sheet's conformance with the Caltrans *Soil & Rock Logging, Classification, and Presentation Manual,* and other applicable standards. To facilitate a quality check, the checker shall be provided with the draft final LOTB sheets, pertinent laboratory test results, copies of approved *Request for Exceptions*, and the field logs. This checklist is not comprehensive and does not attempt to account for all logging and presentation standards. As such, the checker must be familiar with the entire manual in order to successfully perform a quality check. **One checklist shall be completed per LOTB plan sheet. One signature sheet may be used for each structure (Bridge No.).**

Project Information

Dist – EA: 1E4901 County: BUT Route: 32 PM: 11.08

Bridge No.: 12-0135

Sheet Title: Dead Horse Slough Bridge (Widen) Sheets 1 - 5

Revision Date: N/A

Are there approved exceptions to the manual? ☐ Yes ☒ No (attach, if yes)

General

	Yes	No	N/A	
1.1	☒	☐	☐	Does the Plan View meet the requirements of Sec 5.2.3.3?
1.2	☒	☐	☐	Does the Border meet the requirements of Sec 5.2.3.1 and Sec 5.2.3.2?
1.3	☒	☐	☐	Are the Notes clear and do they meet the requirements of Sec 5.2.2?
1.4	☒	☐	☐	If As-Built LOTB, does it meet the requirements of Sec 5.2.4?
1.5	☒	☐	☐	Is the soil legend sheet attached and properly labeled?
1.6	☒	☐	☐	If rock is presented, is the rock legend attached and properly labeled?
1.7	☐	☐	☒	If approved "Exception to Policy" form is attached, does the LOTB meet the requirements of the approved exceptions?

Elevation View

2.1	☒	☐	☐	Are the Hole Identifications correct? (Sec 2.3) (Sec. 5.2.3.4)
2.2	☒	☐	☐	Are the location descriptions correct?
2.3	☒	☐	☐	Are the holes located properly on the profile?
2.4	☒	☐	☐	Is the elevation scale correct? (Sec 5.2.3.4)
2.5	☒	☐	☐	Is the top of hole elevation presented and correct? (Sec 5.2.3.4)

Log of Test Boring (LOTB) Sheet Checklist

Bridge No.: 12-0135 Sheet Title: Dead Horse Slough Bridge (Widen)

	Yes	No	N/A	
2.6	☒	☐	☐	Is the correct hole diameter presented in the correct Borehole Symbol? (Sec 5.2.5.6)
2.7	☒	☐	☐	Does the stationing match the profile view?
2.8	☒	☐	☐	Are the Boring Date and Termination Elevation presented at the bottom of each boring log? (Sec 5.2.3.4)
2.9	☒	☐	☐	If SPT tests were performed, is the correct hammer efficiency reported at the bottom of each borehole?
2.10	☒	☐	☐	Are lab tests reported at the correct elevations? (Sec 5.2.5.2)
2.11	☒	☐	☐	Are SPT blow counts reported at the correct elevations? (Sec 5.2.5.2)
2.12	☒	☐	☐	Is the groundwater presented at the correct elevation? (Sec 5.2.5.2)
2.13	☒	☐	☐	Are the soil/rock layers and graphics presented correctly? (Sec 4, Sec 5.2.5.7)
2.14	☒	☐	☐	Are the required descriptors presented and in the correct order? (Sec 2.4.1, Sec 2.5.1)
2.15	☒	☐	☐	Are the descriptors presented consistent with those allowed in the manual?
2.16	☒	☐	☐	Are the soil identifications consistent with the field observations? (Sec 2)
2.17	☒	☐	☐	Are the soil classifications consistent with reported lab test results? (Sec 3)
2.18	☒	☐	☐	Are the consistency descriptors consistent with field observations and/or lab test results? (Sec 2.4.3, Sec 3.2.3)
2.19	☒	☐	☐	Are the apparent density descriptors consistent with the SPT results and hammer efficiency? (Sec 2.4.4)
2.20	☒	☐	☐	Are % recovery (REC) and rock quality designation (RQD) presented at the required elevations?
2.21	☐	☐	☒	Is rock strength presented where lab tests are reported? (Sec 3.3.1)
2.22	☒	☐	☐	Considering the field observations, are lab test results properly applied to the descriptors within a layer per Sec 4.3?
2.23	☒	☐	☐	Are the presentations consistent with the rules presented in Sec 4?
2.24	☒	☐	☐	Are the presentations consistent with the rules presented in Sec 5?

List all variances identified during initial review of the LOTB sheet and steps needed to resolve the discrepancy (include item number). Also note any recommendations for revisions to the manual or procedures that might reduce or eliminate similar errors in the future.

N/A

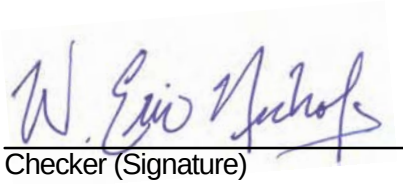
Log of Test Boring QC/QA Signature Sheet

Dist – EA: 1E4901 Bridge No.: 12-0135

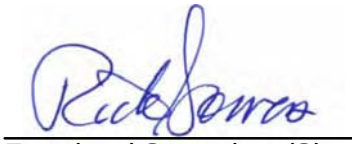
Sheet Titles:

Dead Horse Slough Bridge (Widen) Sheets 1 - 5

I, the undersigned on the date following my signature, hereby certify that I have performed a quality check of the referenced LOTB sheets and that the referenced LOTB sheets *substantially* comply with the Caltrans Soil and Rock Logging, Classification and Presentation Manual (June 2007) and related policy and standards.

Eric Nichols	Senior Project Manager
Checker (Print)	Title
	9-22-2010
Checker (Signature)	Date

I, the undersigned on the date following my signature, hereby certify that the referenced LOTB sheets *substantially* comply with Geotechnical Service's Quality Control/Quality Assurance procedures, as described in the memorandum, "Quality Control/Quality Assurance Documentation on LOTB Sheets", dated July 1, 2007.

Rick Sowers	Senior Project Manager
Functional Supervisor (Print)	Title
	9-22-2010
Functional Supervisor (Signature)	Date

(This original checklist and signature sheet shall be placed in the geotechnical project file, and a copy sent to the Geotechnical Services Corporate Unit (Mark Willian))

APPENDIX B

Laboratory Results

Project Name: Dead Horse Slough Bridge (Widen)

BCI File No: 1202.1

Date: 4/22/2010

Technician: KLC

blackburn consulting

MOISTURE-DENSITY TESTS

Sample No.	A-10-B1-2c	A-10-B1-4c	A-10-B1-6c	A-10-B1-8c	A-10-B1-9c	A-10-B1-11c	A-10-B2-2c
Depth (ft.)	6.0-6.5	16.0-16.5	26.0-26.5	36.0-36.5	41.0-41.5	50.0-50.5	6.0-6.5
Sample Length (in.)	5.82	5.90	5.90	5.79	5.87	5.91	5.87
Diameter (in.)	2.398	2.398	2.421	2.412	2.409	2.406	2.398
Sample + Tube (g)	1112.2	1031.1	1014.3	1028.8	1061.7	1048.2	1089.8
Tube (g)	290.6	289.4	289.7	287.7	289.7	290.2	198.7
Wet Weight (g)	821.6	741.7	724.6	741.1	772.0	758.0	891.1
Sample Volume (ft^3)	0.01522	0.01540	0.01571	0.01531	0.01548	0.01556	0.01535
Tare No.	J	B2	T9	F3	Y	D	T3
Tare (g)	157.6	156.1	257.4	154.5	155.2	167.7	256.8
Wet Soil + Tare (g)	978.5	895.2	981.5	894.5	926.7	920.4	1144.6
Dry Soil + Tare (g)	804.3	687.4	709.0	656.9	676.4	750.0	991.5
Dry Soil Weight (g)	646.7	531.3	451.6	502.4	521.2	582.3	734.7
Water (g)	174.2	207.8	272.5	237.6	250.3	170.4	153.1
Moisture (%)	**26.9**	**39.1**	**60.3**	**47.3**	**48.0**	**29.3**	**20.8**
Wet Density (pcf)	**119.0**	**106.2**	**101.7**	**106.7**	**110.0**	**107.4**	**128.0**
Dry Density (pcf)	**93.8**	**76.3**	**63.4**	**72.5**	**74.3**	**83.1**	**105.9**
Pocket Pen (tsf)	**3.0/4.0**	**0.2**	**2.0/2.5**	**4.0**	**4.5+**	**0.1**	**1.5**

Sample No.	A-10-B2-3c	A-10-B2-7C	A-10-B2-9c				
Depth (ft.)	10.0-10.5	31.0-31.5	41.0-41.5				
Sample Length (in.)	5.63	5.38	5.85				
Diameter (in.)	2.401	2.385	2.398				
Sample + Tube (g)	975.1	956.4	1040.3				
Tube (g)	251.0	274.2	288.3				
Wet Weight (g)	724.1	682.2	752.0				
Sample Volume (ft^3)	0.01474	0.01391	0.01529				
Tare No.	F	XX9	D2				
Tare (g)	154.2	246.7	155.3				
Wet Soil + Tare (g)	871.8	598.5	906.3				
Dry Soil + Tare (g)	786.0	468.5	655.4				
Dry Soil Weight (g)	631.8	221.8	500.1				
Water (g)	85.8	130.0	250.9				
Moisture (%)	**13.6**	**58.6**	**50.2**				
Wet Density (pcf)	**108.3**	**108.1**	**108.4**				
Dry Density (pcf)	**95.4**	**68.2**	**72.2**				
Pocket Pen (tsf)	**N/A**	**3.0**	**2.5/3.5**				

Diameter = 1.44" for 1.5-inch Tubes

Diameter = 1.938" for 2-inch Tubes

Diameter = 2.438" for 2.5-inch Tubes

Diameter= 2.850" for 3.0-inch Shelby Tubes

Unconfined Compression Test
ASTM D 2166-06

blackburn consulting

Project Name: SR 32 Widening
Project Number: 1202.1
Sample: A-10-B1, 3c Depth: 11-11.5
Sample Description: SANDY SILT, dark yellowish brown
Date: 4/21/2010
Tested By: BWM

Original Sample Length	5.98
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

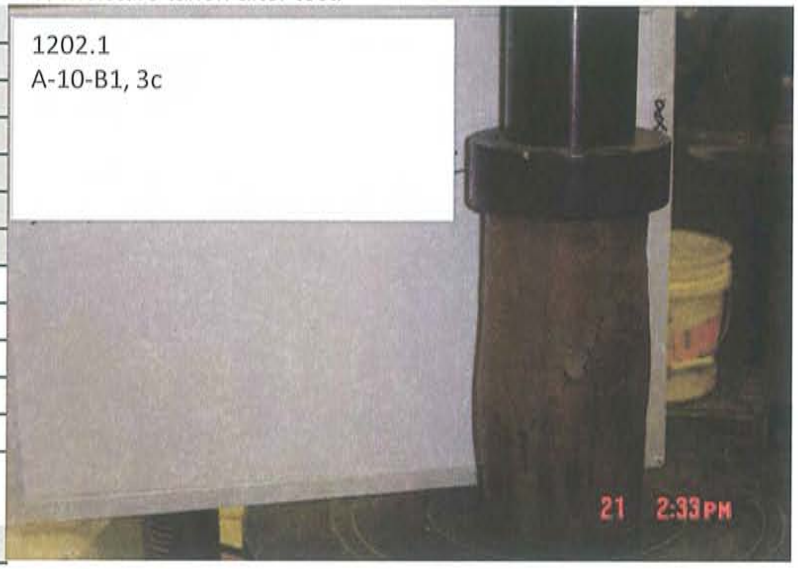
Axial Strain at Max. Load	4.3%
Average cross-sectional area (in^2)	4.73
Deflection at Max. Load (in)	0.259
Maximum Load (lbs)	22
Strain at Failure (%)	1.55
Compressive Strength (tsf)	**0.33**

Moisture Density

Tube and Sample (g)	947.90
Tube (g)	0.00
Sample Weight (g)	947.90
Tare Number	A1
Tare Weight (g)	154.80
Wet Weight (g)	588.40
Dry Weight (g)	469.90
Dry Weight (g)	315.10
Water Weight (g)	118.50
Percent Moisture (%)*	37.6
Wet Density (pcf)	133.5
Dry Density (pcf)	97.0

Remarks:

* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

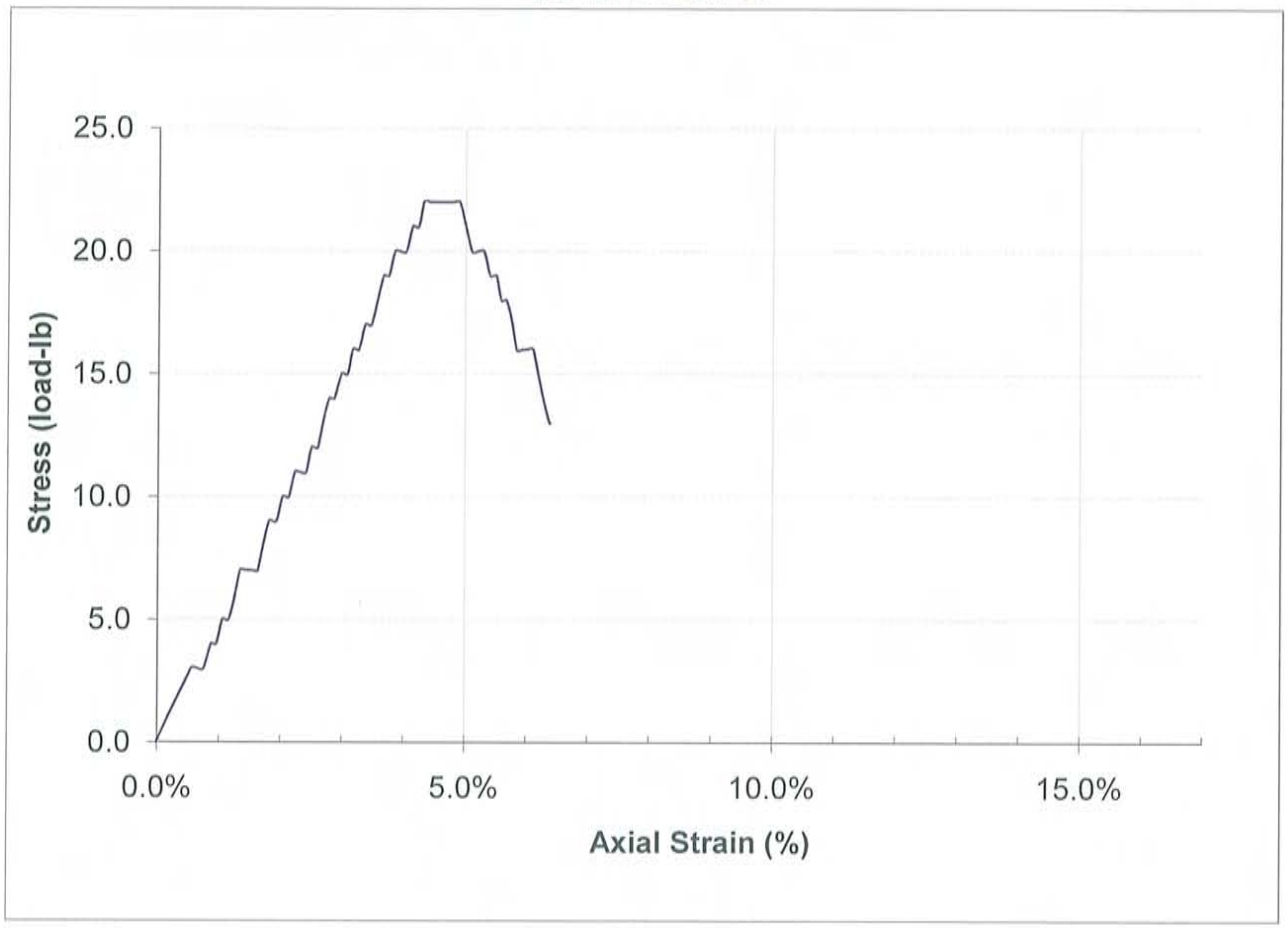
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.034	3	0.128	10	0.220	19	0.312	20
0.040	3	0.134	11	0.225	19	0.318	20
0.046	3	0.139	11	0.231	20	0.324	19
0.053	4	0.145	11	0.236	20	0.330	19
0.058	4	0.150	12	0.242	20	0.335	18
0.064	5	0.156	12	0.248	21	0.340	18
0.070	5	0.161	13	0.254	21	0.346	17
0.076	6	0.167	14	0.259	22	0.350	16
0.081	7	0.172	14	0.266	22	0.356	16
0.087	7	0.179	15	0.271	22	0.360	16
0.093	7	0.185	15	0.277	22	0.366	16
0.098	7	0.190	16	0.282	22	0.371	15
0.103	8	0.196	16	0.288	22	0.376	14
0.109	9	0.202	17	0.294	22	0.382	13
0.116	9	0.208	17	0.300	21	0.383	13
0.122	10	0.214	18	0.306	20		

Project
SR 32 Widening
Project Number
1202.1
Sample Number
A-10-B1, 3c
Material Description
SANDY SILT, dark yellowish brown
Tested By
BWM

ASTM D 2166-06



Wet Density (pcf)	133.5
Dry Density (pcf)	97.0
% Moisture	37.6

Unconfined Compressive Strength (tsf)	0.33

Unconfined Compression Test
ASTM D 2166-06

blackburn consulting

Project Name: SR 32 Widening
Project Number: 1202.1
Sample: A-10-B1, 7c Depth: 31-31.5
Sample Description: SILTY SAND, dark yellowish brown
Date: 4/21/2010
Tested By: BWM

Original Sample Length	5.91
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

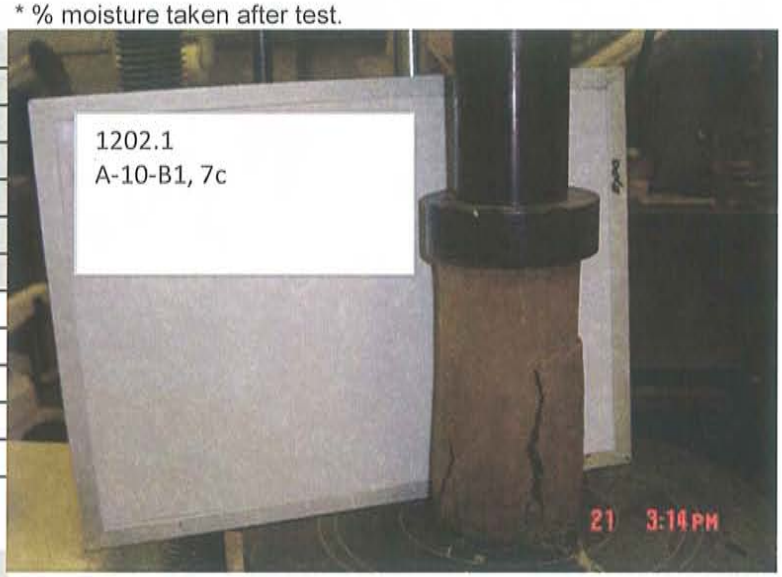
Axial Strain at Max. Load	6.0%
Average cross-sectional area (in^2)	4.81
Deflection at Max. Load (in)	0.354
Maximum Load (lbs)	61
Strain at Failure (%)	2.09
Compressive Strength (tsf)	**0.91**

Moisture Density

Tube and Sample (g)	1027.40
Tube (g)	288.80
Sample Weight (g)	738.60
Tare Number	B1
Tare Weight (g)	154.00
Wet Weight (g)	491.70
Dry Weight (g)	370.40
Dry Weight (g)	216.40
Water Weight (g)	121.30
Percent Moisture (%)*	56.1
Wet Density (pcf)	105.2
Dry Density (pcf)	67.4

Remarks:

* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

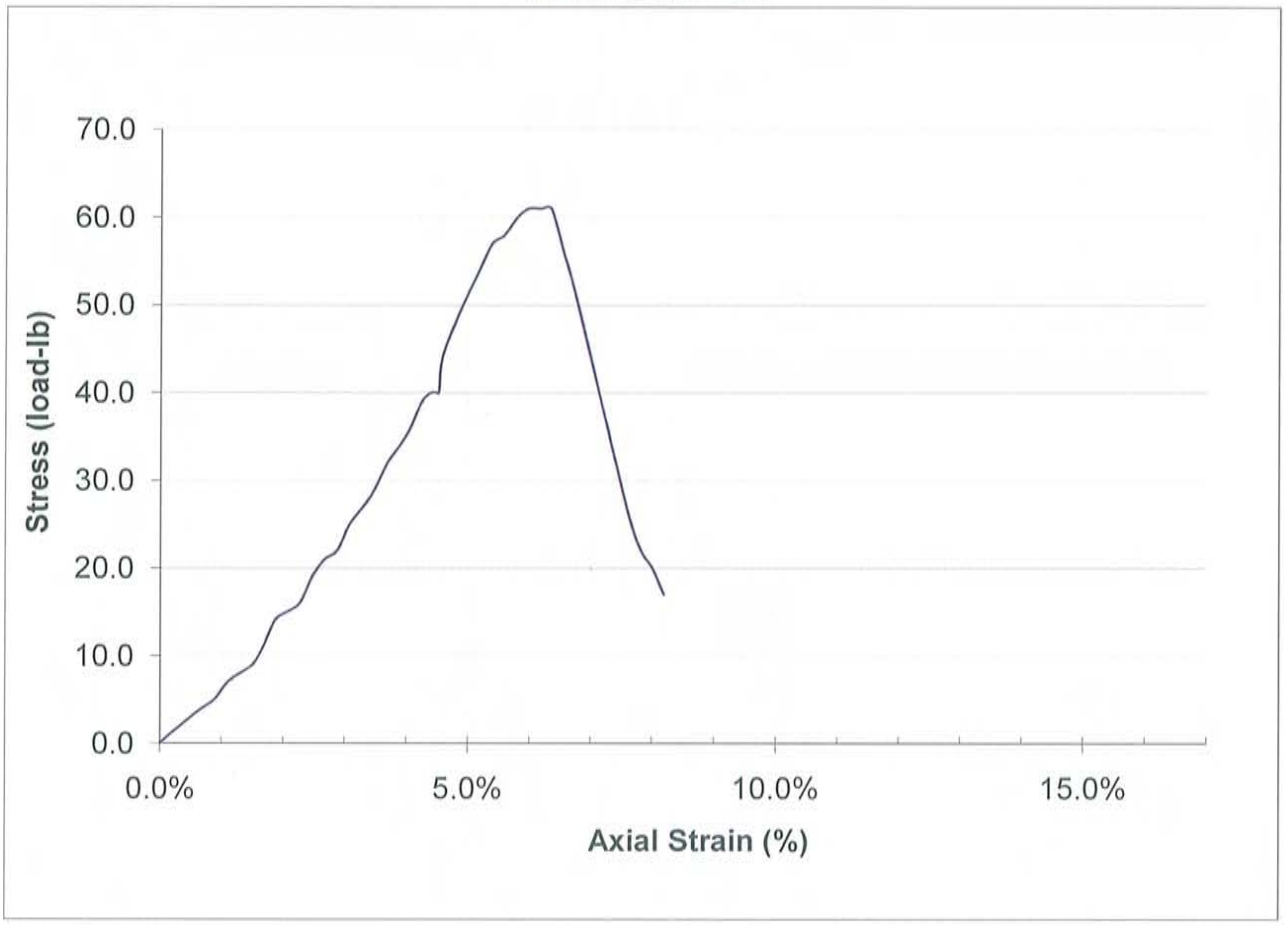
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.030	3	0.219	32	0.366	61		
0.041	4	0.231	34	0.376	61		
0.053	5	0.241	36	0.388	56		
0.066	7	0.252	39	0.400	51		
0.078	8	0.261	40	0.451	26		
0.090	9	0.266	40	0.463	22		
0.100	11	0.268	40	0.474	20		
0.111	14	0.270	43	0.485	17		
0.123	15	0.274	45	0.489	15		
0.135	16	0.284	48	0.490	16		
0.147	19	0.295	51	0.490	15		
0.159	21	0.307	54				
0.171	22	0.319	57				
0.183	25	0.331	58				
0.196	27	0.343	60				
0.207	29	0.354	61				

Project
SR 32 Widening
Project Number
1202.1
Sample Number
A-10-B1, 7c
Material Description
SILTY SAND, dark yellowish brown
Tested By
BWM

ASTM D 2166-06



Wet Density (pcf)	105.2
Dry Density (pcf)	67.4
% Moisture	56.1

Unconfined Compressive Strength (tsf)	0.91

Unconfined Compression Test
ASTM D 2166-06

blackburn consulting

Project Name: SR 32 Widening
Project Number: 1202.1
Sample: A-10-B2, 6c Depth: 26-26.5
Sample Description: Lean CLAY, dark yellowish brown
Date: 4/23/2010
Tested By: BWM

Original Sample Length	5.99
Original Diameter (in)	2.41
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.56

Test Results

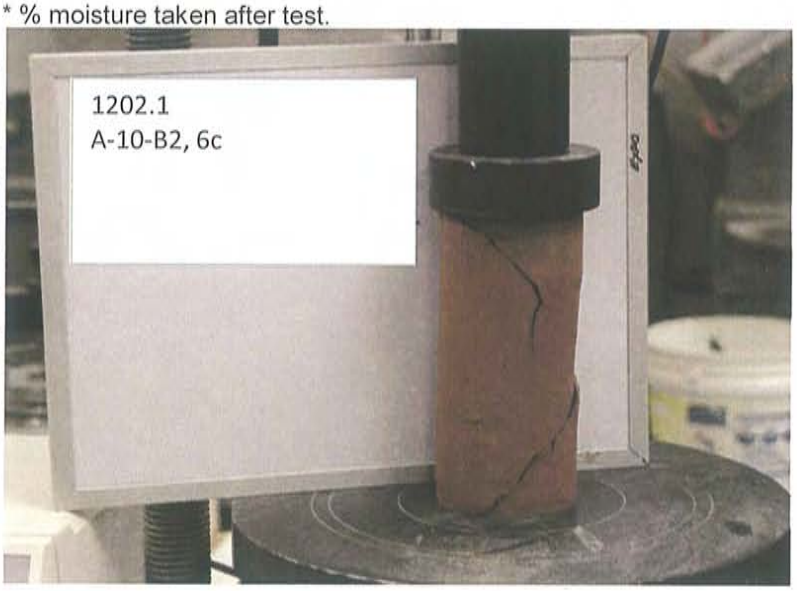
Axial Strain at Max. Load	2.8%
Average cross-sectional area (in^2)	4.69
Deflection at Max. Load (in)	0.165
Maximum Load (lbs)	17
Strain at Failure (%)	0.99
Compressive Strength (tsf)	**0.26**

Moisture Density

Tube and Sample (g)	1017.70
Tube (g)	272.60
Sample Weight (g)	745.10
Tare Number	B3
Tare Weight (g)	151.50
Wet Weight (g)	570.10
Dry Weight (g)	416.90
Dry Weight (g)	265.40
Water Weight (g)	153.20
Percent Moisture (%)*	57.7
Wet Density (pcf)	103.9
Dry Density (pcf)	65.9

Remarks:

* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

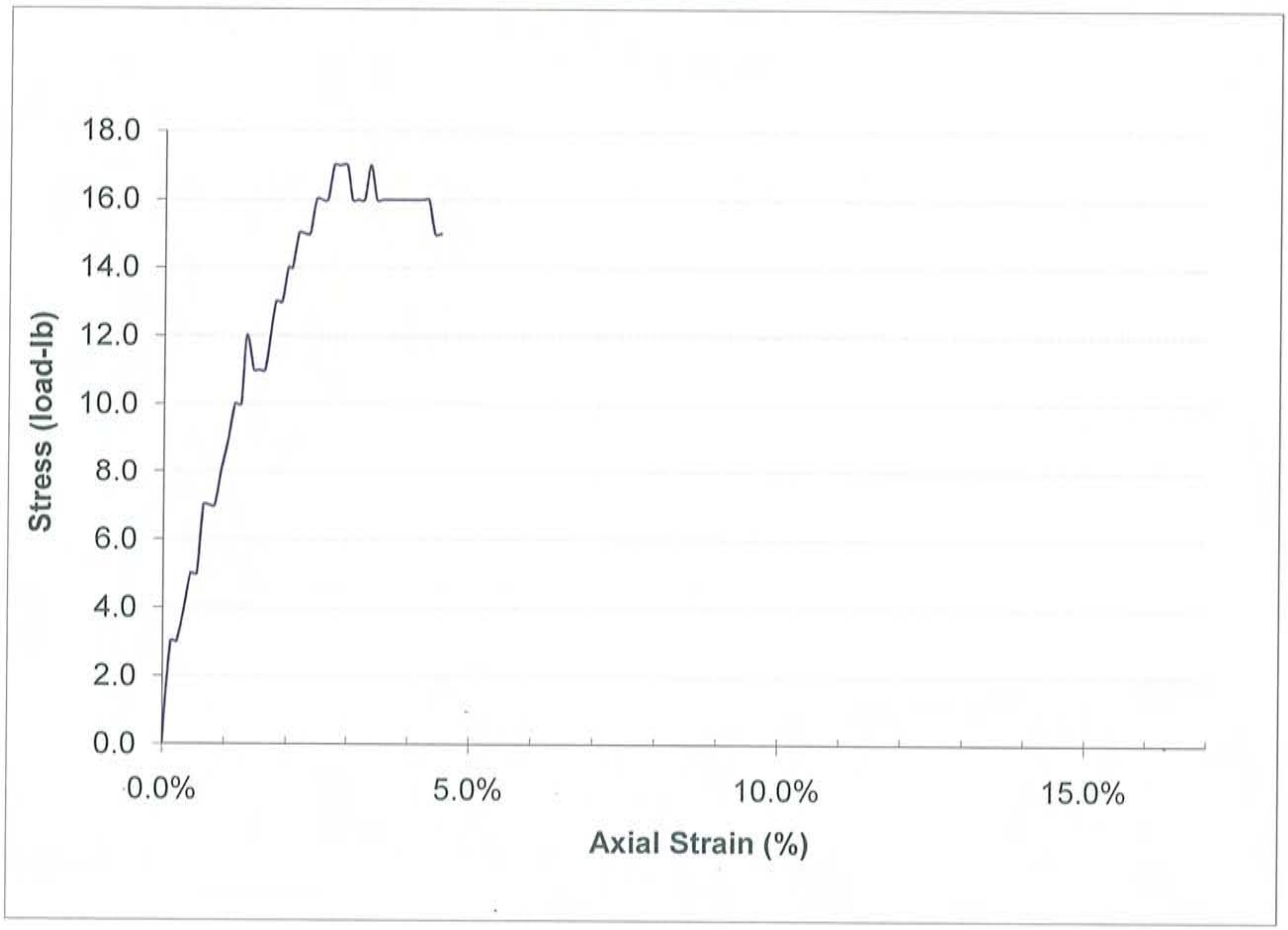
Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.008	3	0.103	12	0.195	16		
0.014	3	0.108	13	0.201	17		
0.021	4	0.114	13	0.207	16		
0.027	5	0.120	14	0.213	16		
0.033	5	0.124	14	0.219	16		
0.039	7	0.130	15	0.225	16		
0.045	7	0.136	15	0.230	16		
0.050	7	0.141	15	0.237	16		
0.056	8	0.147	16	0.242	16		
0.063	9	0.153	16	0.248	16		
0.069	10	0.159	16	0.253	16		
0.075	10	0.165	17	0.258	16		
0.080	12	0.171	17	0.264	15		
0.087	11	0.178	17	0.270	15		
0.092	11	0.183	16	0.276	15		
0.098	11	0.189	16	0.277	15		

Project
SR 32 Widening
Project Number
1202.1
Sample Number
A-10-B2, 6c
Material Description
Lean CLAY, dark yellowish brown
Tested By
BWM

blackburn consulting

ASTM D 2166-06



Wet Density (pcf)	103.9
Dry Density (pcf)	65.9
% Moisture	57.7

Unconfined Compressive Strength (tsf)	0.26

Unconfined Compression Test
ASTM D 2166-06

blackburn consulting

Project Name: SR 32
Project Number: 1202.1
Sample: A-10-B2, 8c Depth: 36-36.5
Sample Description: Lean CLAY, dark yellowish brown, partially cemented
Date: 4/23/2010
Tested By: BWM

Original Sample Length	5.95
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

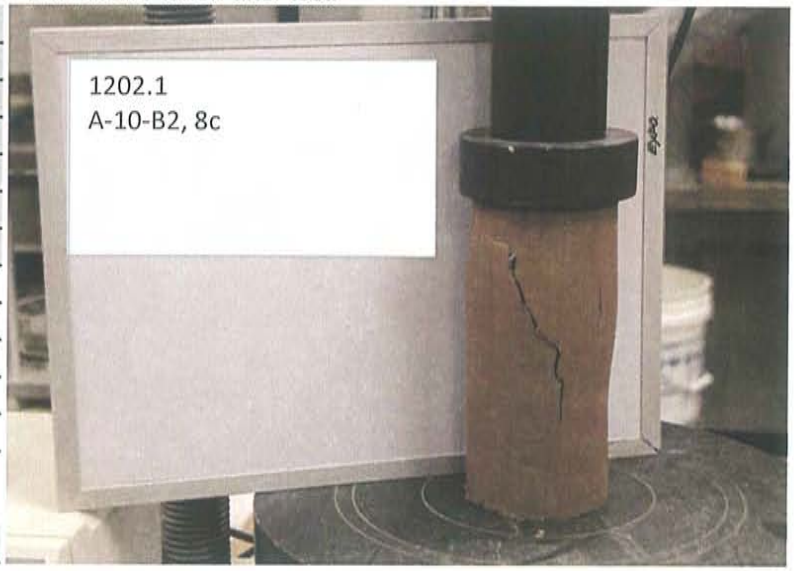
Axial Strain at Max. Load	6.9%
Average cross-sectional area (in^2)	4.86
Deflection at Max. Load (in)	0.409
Maximum Load (lbs)	115
Strain at Failure (%)	2.43
Compressive Strength (tsf)	**1.70**

Moisture Density

Tube and Sample (g)	1047.30
Tube (g)	287.10
Sample Weight (g)	760.20
Tare Number	B4
Tare Weight (g)	152.90
Wet Weight (g)	569.30
Dry Weight (g)	428.20
Dry Weight (g)	275.30
Water Weight (g)	141.10
Percent Moisture (%)*	51.3
Wet Density (pcf)	107.6
Dry Density (pcf)	71.1

Remarks:

* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

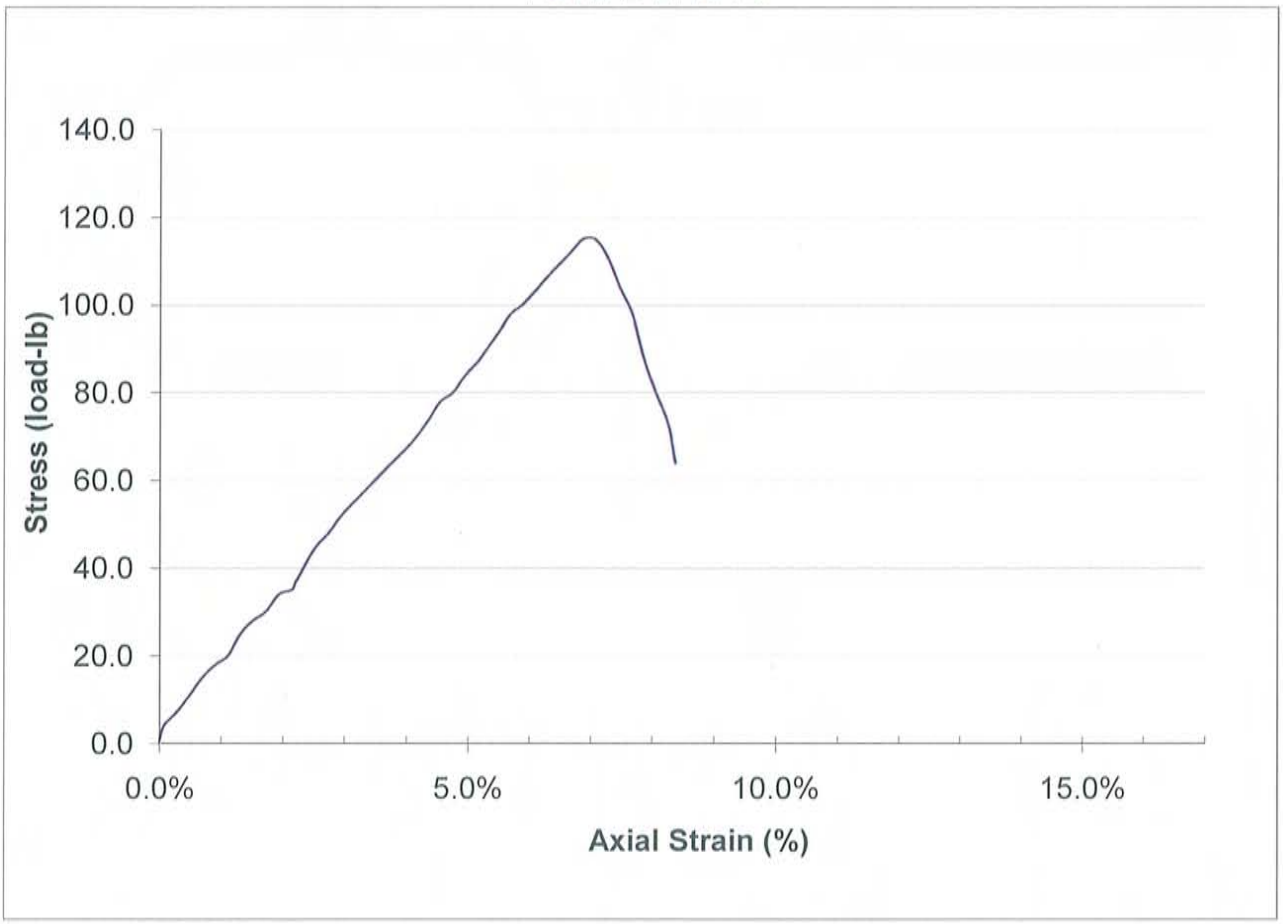
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.005	4	0.164	48	0.350	100		
0.017	7	0.176	52	0.362	103		
0.030	11	0.188	55	0.373	106		
0.042	15	0.200	58	0.385	109		
0.055	18	0.212	61	0.397	112		
0.067	20	0.224	64	0.409	115		
0.079	25	0.237	67	0.421	115		
0.091	28	0.248	70	0.433	111		
0.103	30	0.260	74	0.445	104		
0.116	34	0.271	78	0.457	98		
0.128	35	0.283	80	0.468	88		
0.131	36	0.295	84	0.480	80		
0.132	37	0.307	87	0.492	73		
0.133	37	0.319	91	0.499	64		
0.142	41	0.328	94				
0.152	45	0.339	98				

Project
SR 32
Project Number
1202.1
Sample Number
A-10-B2, 8c
Material Description
Lean CLAY, dark yellowish brown, partially cemented
Tested By
BWM

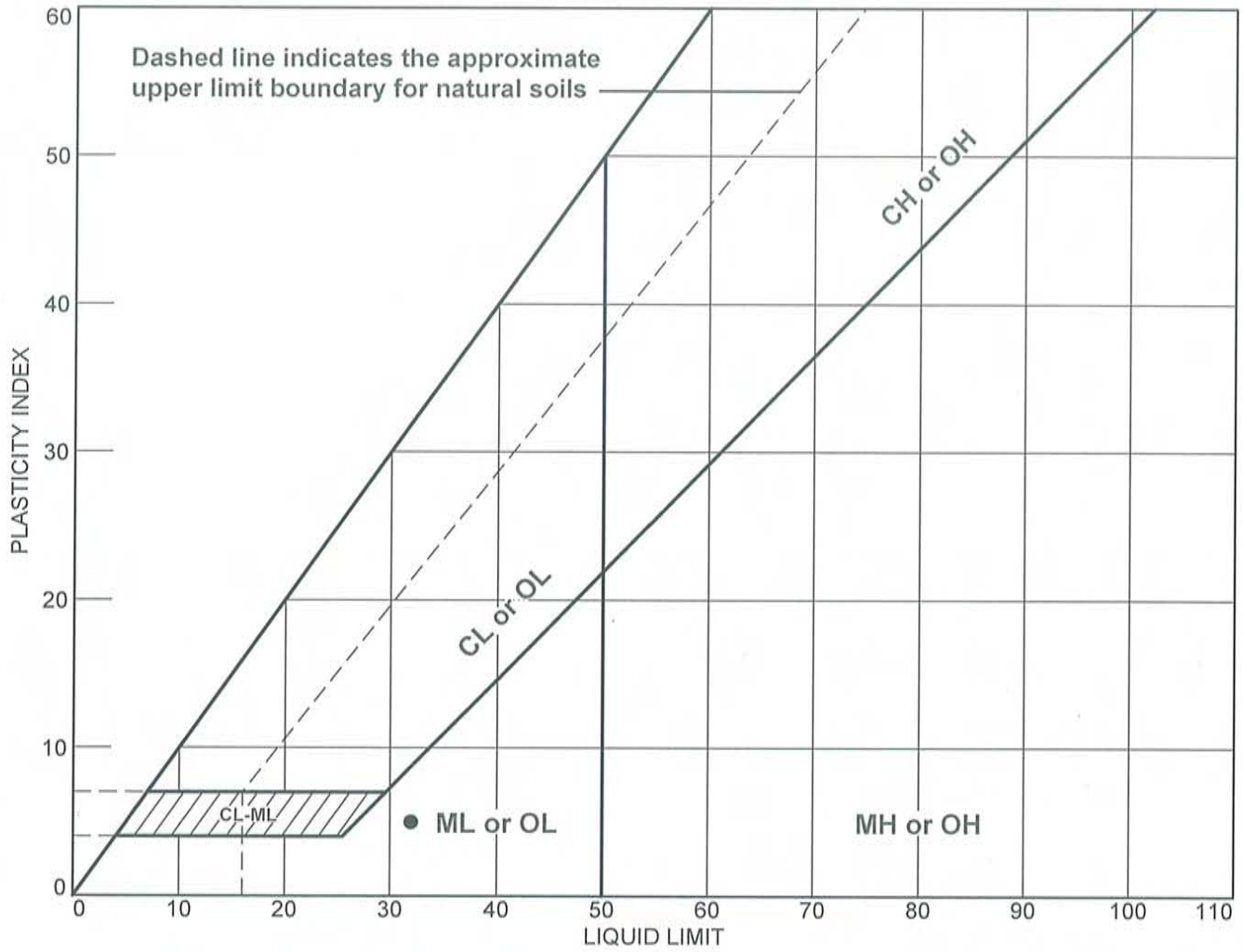
ASTM D 2166-06



Wet Density (pcf)	107.6
Dry Density (pcf)	71.1
% Moisture	51.3

Unconfined Compressive Strength (tsf)	1.70

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	A-10-B1	B1-3c	11.0-11.5		27	32	5	ML
■	A-10-B1	B1-7C	31.0-31.5		NP	NV	NP	ML

Blackburn Consulting

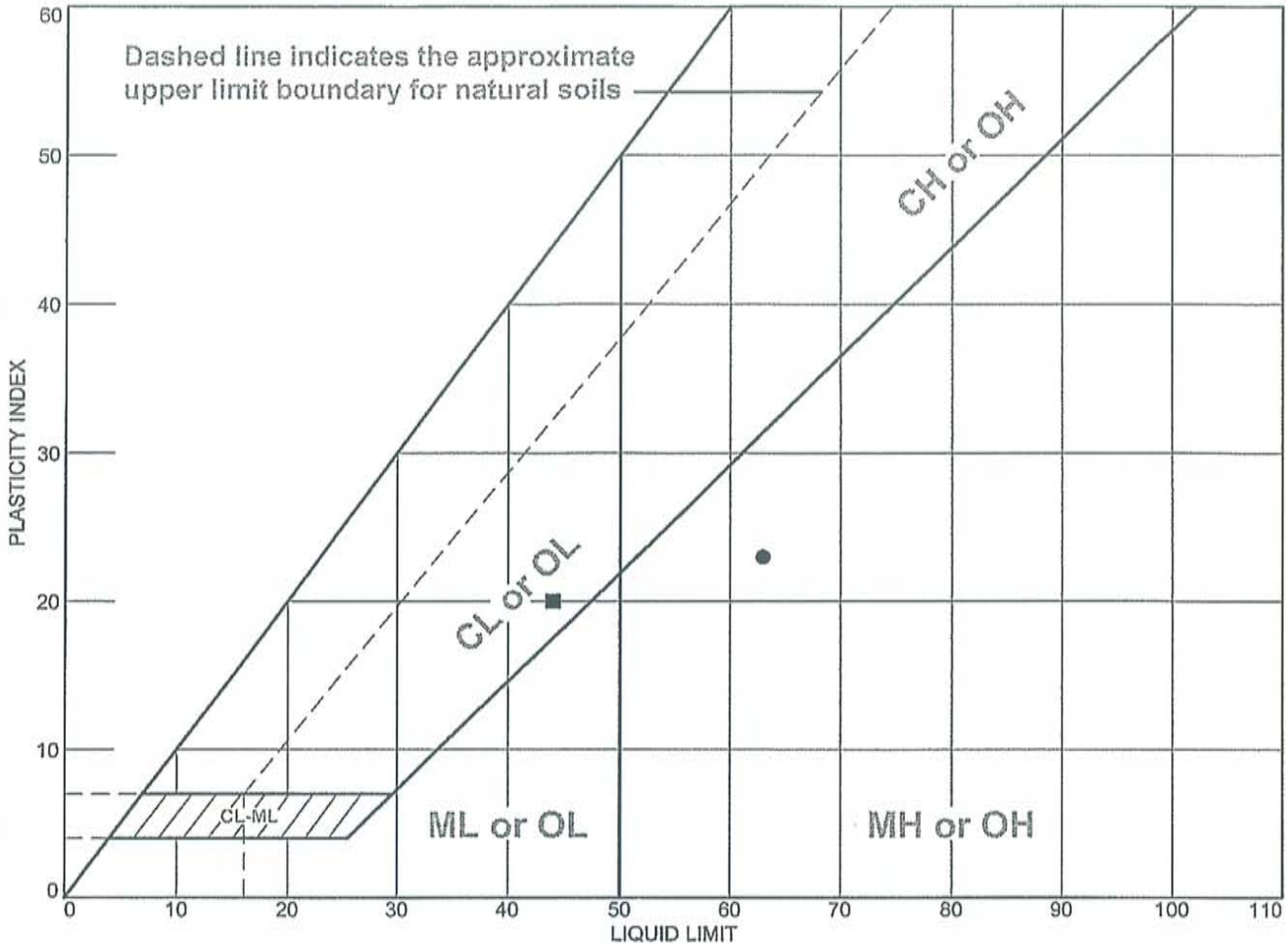
W. Sacramento, CA

Client: Mark Thomas & Company, Inc.
Project: SR 32 Widening

Project No.: 1202.1 **Figure**

Tested By: MAR **Checked By:** MAR

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Strong Brown Elastic SILT	63	40	23			MH
■	Very Dark Brown Lean CLAY	44	24	20			CL

Project No. 1202.1 **Client:** Mark Thomas Company

Project: SR 32 Widening

● **Depth:** 31.0'-31.5' **Sample Number:** A-10-B2-7C

■ **Depth:** 5.5'-6.0' **Sample Number:** A-10-B1-2B

Remarks:

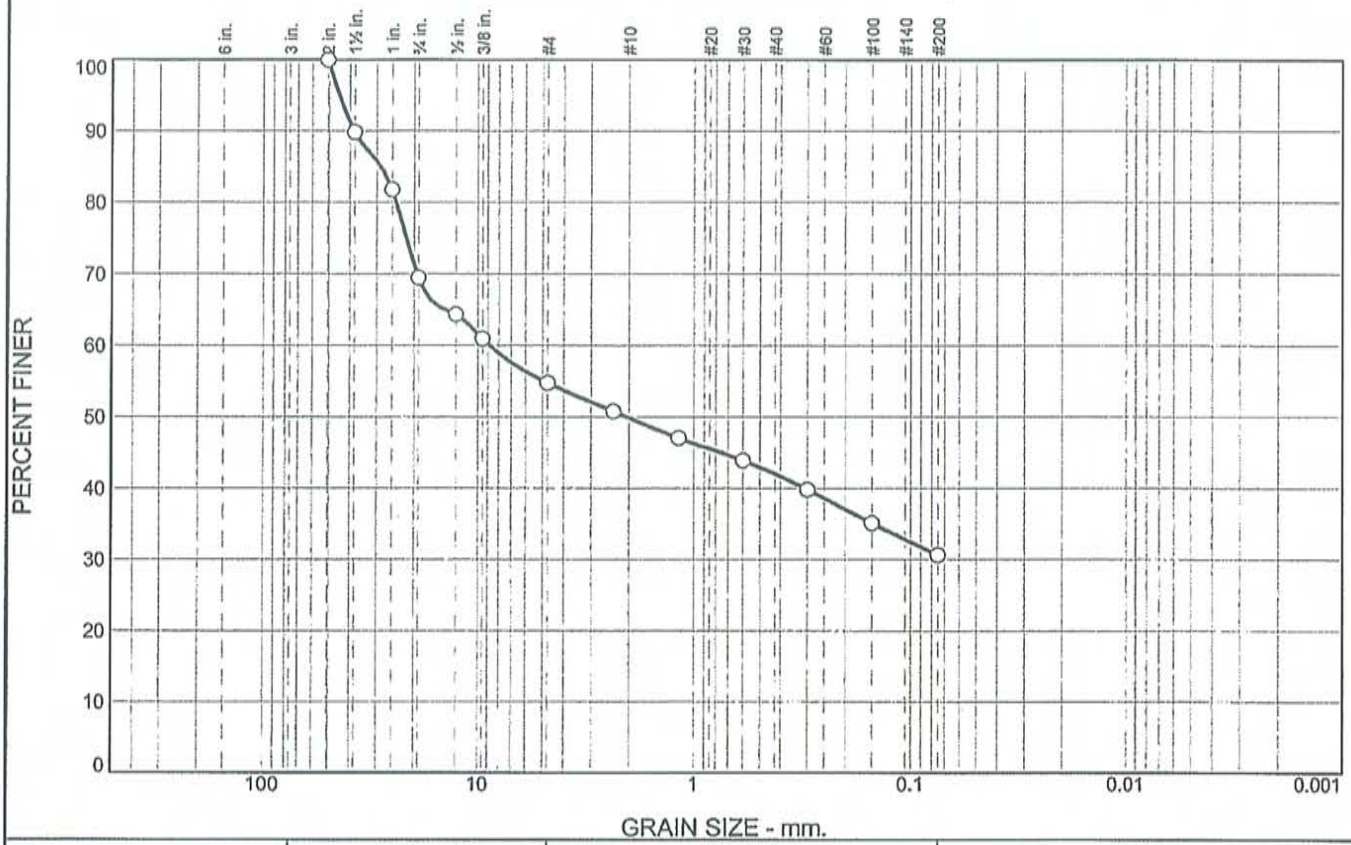
Blackburn Consulting

Auburn, CA

Figure

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.5	14.7	4.9	7.9	11.4	30.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	89.8		
1"	81.8		
3/4"	69.5		
1/2"	64.3		
3/8"	60.9		
#4	54.8		
#8	50.8		
#16	47.1		
#30	43.9		
#50	39.8		
#100	35.1		
#200	30.6		

* (no specification provided)

Material Description

Brown Clayey Gravel with Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 38.3353 D_{85}= 28.6615 D_{60}= 8.8214
D_{50}= 2.0415 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GC AASHTO=

Remarks

Sample Number: A10-B1 5C **Depth:** 20.5'-20.9'

Date: 5/3/10

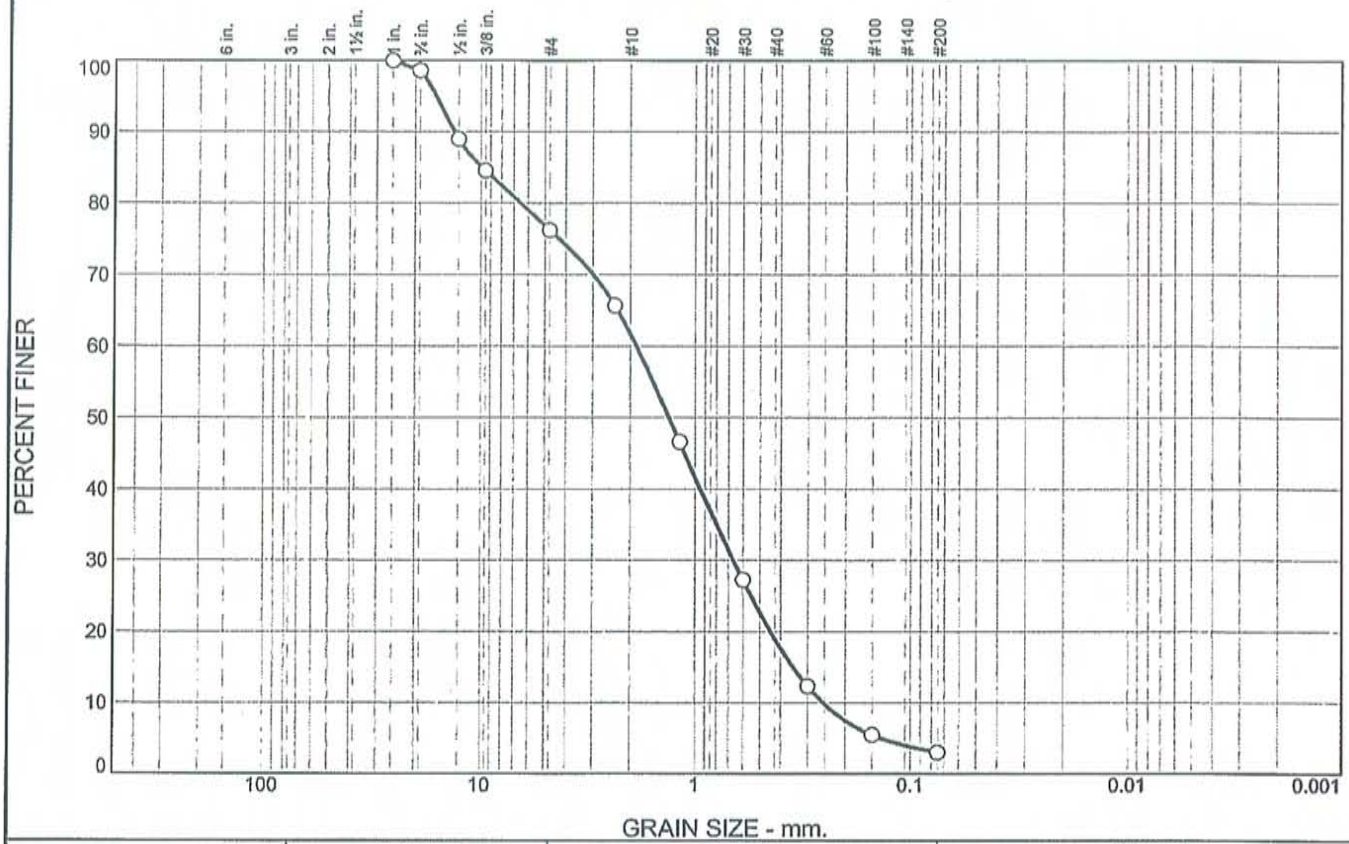
Blackburn Consulting

Auburn, CA

Client: Mark Thomas Company
Project: SR 32 Widening
Project No: 1202.1 **Figure**

Tested By: DKB **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	1.4	22.4	14.4	42.9	15.9	3.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	98.6		
1/2"	89.0		
3/8"	84.6		
#4	76.2		
#8	65.7		
#16	46.6		
#30	27.2		
#50	12.3		
#100	5.5		
#200	3.0		

* (no specification provided)

Material Description

Olive Brown Poorly Graded SAND with Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.2890 D_{85}= 9.8848 D_{60}= 1.8668
D_{50}= 1.3193 D_{30}= 0.6654 D_{15}= 0.3506
D_{10}= 0.2541 C_u= 7.35 C_c= 0.93

Classification

USCS= SP AASHTO=

Remarks

Sample Number: A-10-B2 4 **Depth:** 15.0'-16.5'

Date: 5/3/10

Blackburn Consulting

Auburn, CA

Client: Mark Thomas Company
Project: SR 32 Widening
Project No: 1202.1 **Figure**

Tested By: DKB **Checked By:** KLC

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

Date Reported 04/23/2010
Date Submitted 04/20/2010

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

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

The reported analysis was requested for the following location:
Location : 1202.1/SR 32 WDENING Site ID : [redacted].
Your purchase order number is 1202.1. A-10-B1 6
Thank you for your business.

* For future reference to this analysis please use SUN # 57764-117183.

EVALUATION FOR SOIL CORROSION

Soil pH	6.26	
Minimum Resistivity	1.80 ohm-cm (x1000)	
Chloride	33.4 ppm	00.00334 %
Sulfate	5.9 ppm	00.00059 %

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

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

Date Reported 04/23/2010
Date Submitted 04/20/2010

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

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

The reported analysis was requested for the following location:
Location : 1202.1/SR 32 WDENING Site ID : [redacted]-7B.
Your purchase order number is 1202.1. A-10-B2
Thank you for your business.

* For future reference to this analysis please use SUN # 57764-117184.

EVALUATION FOR SOIL CORROSION

Soil pH	6.28	
Minimum Resistivity	1.72 ohm-cm (x1000)	
Chloride	39.8 ppm	00.00398 %
Sulfate	3.4 ppm	00.00034 %

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

APPENDIX C

Design Acceleration Response Spectra

Caltrans Fault and Site Data Input Sheet

Caltrans ARS Online Data Output

DESIGN ACCELERATION RESPONSE SPECTRA

Dead Horse Slough Bridge (Widen)
BCI No.:1202.1

Envelope T (sec)	Envelope S(a)
0.01	0.226
0.02	0.241
0.022	0.246
0.025	0.254
0.029	0.262
0.03	0.265
0.032	0.268
0.035	0.274
0.036	0.276
0.04	0.282
0.042	0.286
0.044	0.289
0.045	0.29
0.046	0.292
0.048	0.294
0.05	0.297
0.055	0.304
0.06	0.31
0.065	0.323
0.067	0.328
0.07	0.336
0.075	0.35
0.08	0.363
0.085	0.377
0.09	0.39
0.095	0.403
0.1	0.416
0.11	0.436
0.12	0.454
0.13	0.471
0.133	0.475
0.14	0.485
0.15	0.498
0.16	0.504
0.17	0.509
0.18	0.513
0.19	0.516
0.2	0.519
0.22	0.514
0.24	0.508
0.25	0.505
0.26	0.501
0.28	0.494
0.29	0.49
0.3	0.486
0.32	0.476
0.34	0.467
0.35	0.462
0.36	0.457

Envelope T (sec)	Envelope S(a)
0.38	0.447
0.4	0.438
0.42	0.426
0.44	0.415
0.45	0.41
0.46	0.405
0.48	0.395
0.5	0.385
0.55	0.366
0.6	0.35
0.65	0.336
0.667	0.331
0.7	0.323
0.75	0.312
0.8	0.299
0.85	0.287
0.9	0.276
0.95	0.267
1	0.258
1.1	0.239
1.2	0.223
1.3	0.209
1.4	0.197
1.5	0.186
1.6	0.177
1.7	0.168
1.8	0.161
1.9	0.154
2	0.148
2.2	0.132
2.4	0.119
2.5	0.113
2.6	0.108
2.8	0.099
3	0.091
3.2	0.084
3.4	0.077
3.5	0.074
3.6	0.071
3.8	0.066
4	0.062
4.2	0.059
4.4	0.056
4.6	0.053
4.8	0.05
5	0.048

Fault & Site Data Input Sheet

The input sheet is to help the user organize the site data for developing the design response spectrum. Beta-Testers: If you fill out the fault and site information for each location, please provide this document and the *checker* to facilitate the QC/QA procedures listed in the 2009 Deterministic Fault Information & Seismic Procedures QC/QA checklist.

Project Information

Dist – EA: 3 County: BUT Route: 32 PM: 11.08

Bridge/Facility Name: DEAD HORSE SLOUGH Bridge/Facility No.: 12-0135

Latitude: 39.740°N Longitude: 121.803°W

Fault Information (#1)

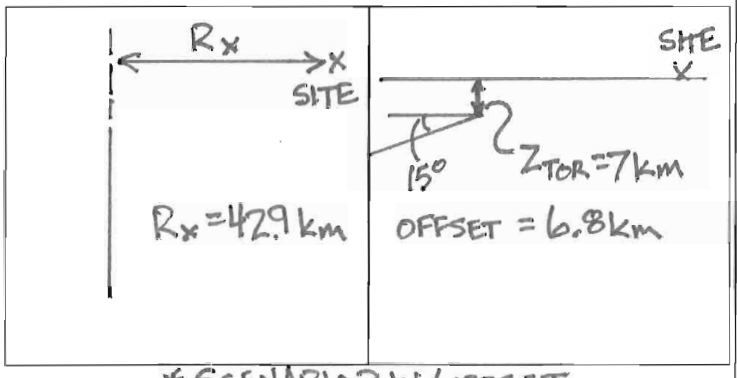
Fault Name: GREAT VALLEY 1 Fault ID#: 20

MMax: 6.7 Fault Type: R

Fault Dip: 15° Dip Direction: W

Top of Rupture: 7km Bottom of Rupture: 10km

Plan View **Elevation View**



* SCENARIO 3 W / OFFSET

Calculated or Measure Distances

R_{RUP}: 44.00 km

R_{JB}: 43.44 km

R_X: 42.90 km

Determination of V_{S30}

V_{S30} (m/s): 325

Determination of Z1.0 and Z2.5

$Z_{1.0}$ (m/s): /

$Z_{2.5}$ (km/s): /

Notes:

Fault Information (#2) (NCS)

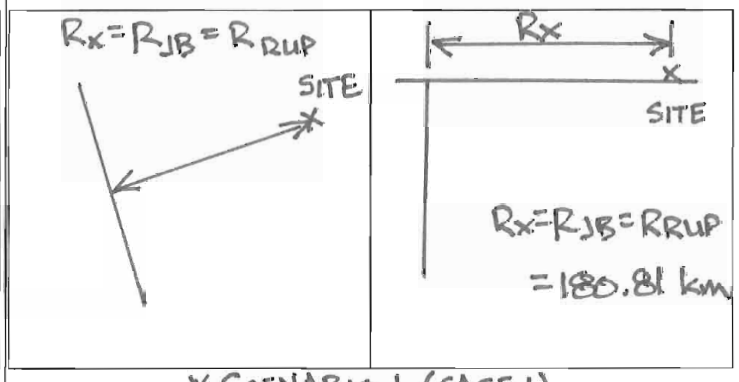
Fault Name: SAN ANDREAS Fault ID#: 308

MMax: 7.9 Fault Type: RLSS

Fault Dip: 90° Dip Direction: V

Top of Rupture: 0km Bottom of Rupture: 11.0km

Plan View **Elevation View**



* SCENARIO 1 (CASE 1)

Calculated or Measure Distances

R_{RUP}: 180.81 km

R_{JB}: 180.81 km

R_X: 180.81 km

Determination of V_{S30}

V_{S30} (m/s): 325

Determination of Z1.0 and Z2.5

$Z_{1.0}$ (m/s): /

$Z_{2.5}$ (km/s): /

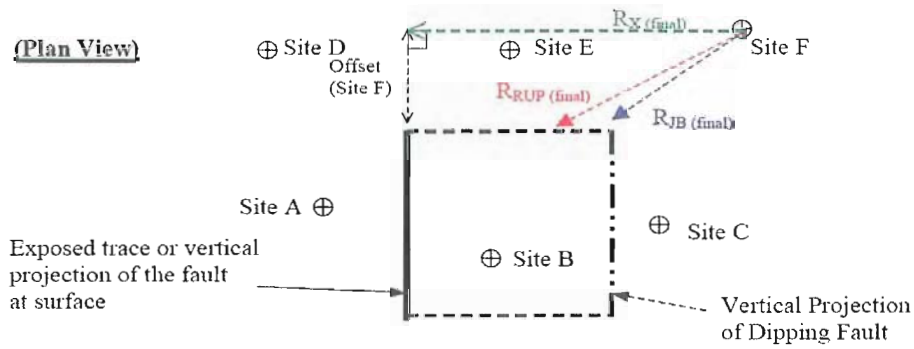
Notes:

Calculating Distances (R_X, R_{RUP}, & R_{JB})

(Caltrans Geotechnical Services Design Manual, Verison 1.0, August 2009)

Project: Dead Horse Slough (Widen)
Fault: Great Valley fault 1
BCI No.: 1202.1

Scenario 3: Offset Corrections for final R_X, R_{JB}, and R_{RUP} distances.



Preliminary Values

Fault dip not vertical and site is on footwall side, $Z_{TOR} > 0$

R_X = Preliminary R_{JB}

Preliminary $R_{RUP} = (Z_{TOR}^2 + R_X^2)^{0.5}$

Z_{TOR} =	7.0	km

R_X =	42.90	km
Preliminary R_{JB} =	42.90	km
Preliminary R_{RUP} =	43.47	km

Final Values

Offset =	6.8	km

Final $R_{JB} = (R_{JB}^2 + \text{Offset}^2)^{0.5}$

Final $R_{RUP} = (R_{RUP}^2 + \text{Offset}^2)^{0.5}$

R_X =	42.90	km
Final R_{JB} =	43.44	km
Final R_{RUP} =	44.00	km

Calculating Distances (R_X, R_{RUP}, & R_{JB})

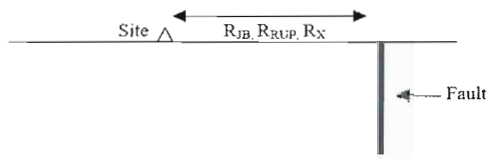
(Caltrans Geotechnical Services Design Manual, Verison 1.0, August 2009)

Project: Dead Horse Slough (Widen)
Fault: San Andreas fault zone (North Coast Section)
BCI No.: 1202.1

Scenario 1 (Case 1): Fault dip = vertical (fault exposed at surface: $Z_{TOR} = 0$)

$R_X = R_{JB} = R_{RUP}$

R_X =	180.81	km
R_{JB} =	180.81	km
R_{RUP} =	180.81	km

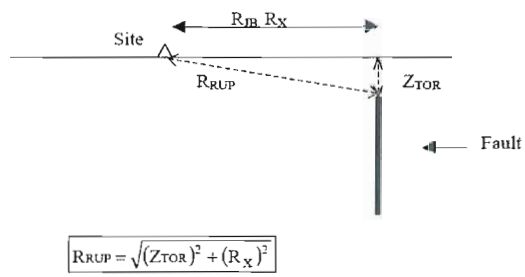


Scenario 1 (Case 2): Fault dip = vertical (buried fault: $Z_{TOR} > 0$)

$R_X = R_{JB}$

$R_{RUP} = (Z_{TOR}^2 + R_X^2)^{0.5}$

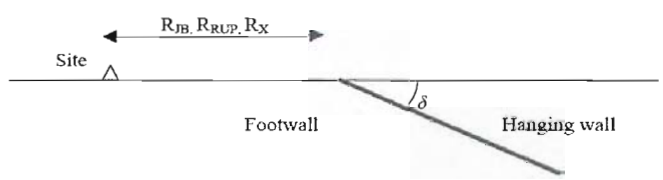
Z_{TOR} =		km
R_X =		km
R_{JB} =	0	km
R_{RUP} =	0	km



Scenario 1 (Case 3): Fault dip not vertical and site is on footwall side, $Z_{TOR} = 0$

$R_X = R_{JB} = R_{RUP}$

R_X =		km
R_{JB} =	0	km
R_{RUP} =	0	km

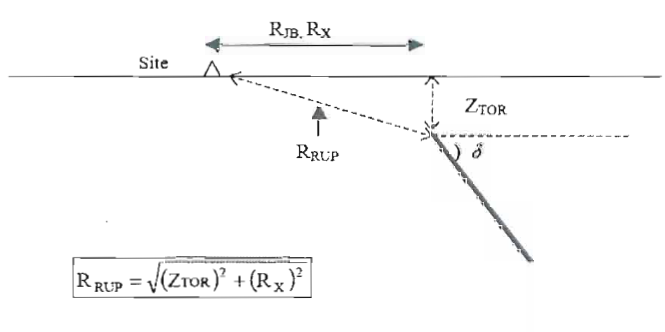


Scenario 1 (Case 4): Fault dip not vertical and site is on footwall side, $Z_{TOR} > 0$

$R_X = R_{JB}$

$R_{RUP} = (Z_{TOR}^2 + R_X^2)^{0.5}$

Z_{TOR} =		km
R_X =		km
R_{JB} =	0	km
R_{RUP} =	0	km



Shear Wave Velocity (Vs)

Project: Dead Horse Slough (Widen)
BCI No: 2102.1
Boring: A-10-B1
Date: 7/28/2010

Hammer Energy (ER): 80 %

Sample Number	Depth (feet)	Sample D_i (inches)	Soil Type	Rock *	Field N (bpf)	N_{SPT} (bpf) **	N_{60}	d Layer Thickness (m)	Vs Granular (m/sec)	Vs Cohesive[1] (m/sec)	Vs Cohesive[2] (m/sec)	Vs Sedimentary Rock (m/sec)	Profile Vs (m/sec)	Profile D/Vs (m/sec)
1	3.0	2.4	CL		25	16	22	0.91			243		243	0.004
2	6.0	2.4	CL		22	14	19	0.91			232		232	0.004
3	11.0	2.4	ML		16	10	14	1.52			209		209	0.007
4	16.0	2.4	ML		10	7	9	1.52			181		181	0.008
5	20.5	2.4	GC		120	78	104	1.37	386				386	0.004
6	26.0	2.4	CL		17	11	15	1.68			214		214	0.008
7	31.0	2.4	ML		29	19	25	1.52			254		254	0.006
8	36.0	2.4	SC		29	19	25	1.52	256				256	0.006
9	41.0	2.4	CL		26	17	23	1.52			247		247	0.006
10	46.0	2.4	GC		67	44	58	1.52	326				326	0.005
11	50.0	2.4	GC		100	65	87	1.22	367				367	0.003
12	50.0	2.4	GC		200	130	173	0.00	448				448	0.000
13	56.0	2.4	GC	rock	100	65	87	1.83	367			453	453	0.004
14	61.0	2.4	GC	rock	100	65	87	1.52	367			453	453	0.003
15	66.0	2.4	GC	rock	100	65	87	1.52	367			453	453	0.003
16	71.0	2.4	GC	rock	100	65	87	1.52	367			453	453	0.003
17	74.0	2.4	GC	rock	100	65	87	0.91	367			453	453	0.002
18	76.0	2.4	GC	rock	100	65	87	0.61	367			453	453	0.001
19	80.0	2.4	GC	rock	100	65	87	1.22	367			453	453	0.003
								24.38					344	0.081

* Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks.
** Corrected for sample diameter

Shear Wave Velocity for Upper 30 m (V_{S30})

$Vs_{30} = [1.45 - (0.015*d)]*Vs(d)$

d (m)	24.38
Vs(d) (m/sec)	301
V_{S30} (m/sec)	**326**

Shear Wave Velocity Correlations

Granular:	$Vs = 100.5(N_{60})^{0.29}$	SPT Correlation
Cohesive[1]:	$Vs = 203(S_u/P_a)^{0.475}$	Undrained Shear Strength (S_u) Correlation
Cohesive[2]:	$Vs = 86.9(N_{60})^{0.333}$	SPT Correlation
Sedimentary Rock:	$Vs = 109(N_{60})^{0.319} \leq 560$/m/sec	SPT Correlation

Note: In the absence of in-situ measurements, limit V_{s30} to 760 m/sec for competent rock in California.

Reference: Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009, pages 15-17.

Shear Wave Velocity (Vs)

Project: Dead Horse Slough (Widen)
BCI No: 2102.1
Boring: A-10-B2
Date: 7/28/2010

Hammer Energy (ER): 80 %

Sample Number	Depth (feet)	Sample D_i (inches)	Soil Type	Rock *	Field N (bpf)	N_{SPT} (bpf) **	N_{60}	d Layer Thickness (m)	Vs Granular (m/sec)	Vs Cohesive[1] (m/sec)	Vs Cohesive[2] (m/sec)	Vs Sedimentary Rock (m/sec)	Profile Vs (m/sec)	Profile D/Vs (m/sec)
1	3.0	2.4	CL		13	8	11	0.91			193		193	0.005
2	6.0	2.4	CL		14	9	12	0.91			199		199	0.005
3	10.0	2.4	CL		100	65	87	1.22			384		384	0.003
4	16.0	1.4	SP		36	36	48	1.83	309				309	0.006
5	21.0	1.4	GC		59	59	79	1.52	357				357	0.004
6	26.0	2.4	MH		13	8	11	1.52			193		193	0.008
7	31.0	2.4	MH		15	10	13	1.52			204		204	0.007
8	36.0	2.4	MH		23	15	20	1.52			236		236	0.006
9	41.0	2.4	MH		20	13	17	1.52			223		223	0.007
10	45.5	2.4	GW		100	65	87	1.37	367				367	0.004
11	51.0	2.4	GW	rock	100	65	87	1.68	367			453	453	0.004
12	55.0	2.4	GW	rock	100	65	87	1.22	367			453	453	0.003
13	57.0	2.4	GW	rock	100	65	87	0.61	367			453	453	0.001
14	60.0	2.4	GW	rock	100	65	87	0.91	367			453	453	0.002
								18.29					320	0.065

* Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks.
** Corrected for sample diameter

Shear Wave Velocity for Upper 30 m (V_{S30})

$Vs_{30} = [1.45 - (0.015*d)]*Vs(d)$

d (m)	18.29
Vs(d) (m/sec)	282
V_{S30} (m/sec)	**332**

Shear Wave Velocity Correlations

Granular: $Vs = 100.5(N_{60})^{0.29}$ SPT Correlation

Cohesive[1]: $Vs = 203(S_u/P_a)^{0.475}$ Undrained Shear Strength (S_u) Correlation

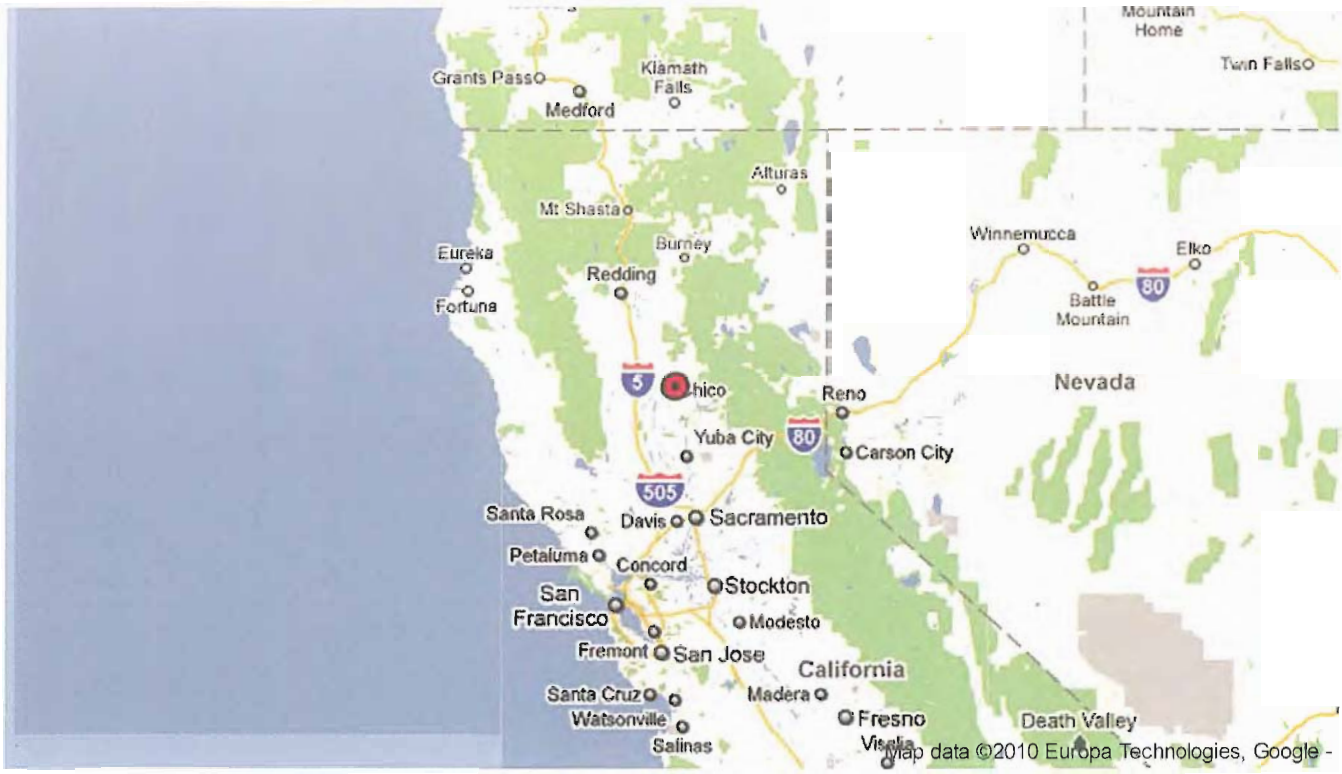
Cohesive[2]: $Vs = 86.9(N_{60})^{0.333}$ SPT Correlation

Sedimentary Rock: $Vs = 109(N_{60})^{0.319} \leq 560$/m/sec SPT Correlation

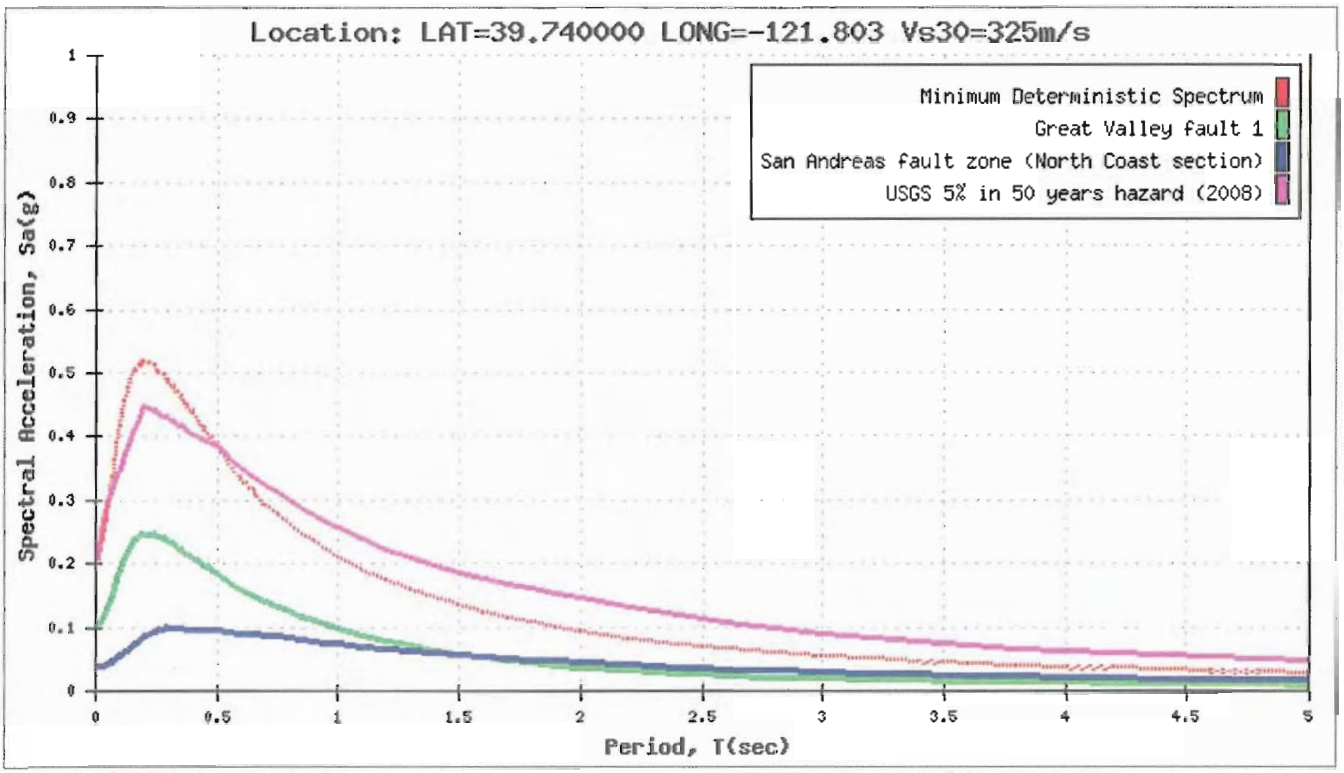
Note: In the absence of in-situ measurements, limit V_{s30} to 760 m/sec for competent rock in California.

Reference: Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009, pages 15-17.

SELECT SITE LOCATION



CALCULATED SPECTRA



SITE DATA

Shear Wave Velocity, Vs_{30}:	325 m/s
Latitude:	39.740000
Longitude:	-121.803000
Depth to Vs = 1.0 km/s:	298 m
Depth to Vs = 2.5 km/s:	2.00 km

DETERMINISTIC

Great Valley fault 1

Fault ID:	20
Maximum Magnitude (MMax):	6.7
Fault Type:	R
Fault Dip:	15 Deg
Dip Direction:	W
Bottom of Rupture Plane:	10.00 km
Top of Rupture Plane(Ztor):	7.00 km
Rrup	43.83 km
Rjb:	43.27 km
Rx:	42.92 km
Fnorm:	0
Frev:	1

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.108	1.000	1.000	0.108
0.02	0.109	1.000	1.000	0.109
0.022	0.110	1.000	1.000	0.110
0.025	0.112	1.000	1.000	0.112
0.029	0.113	1.000	1.000	0.113
0.03	0.114	1.000	1.000	0.114
0.032	0.115	1.000	1.000	0.115
0.035	0.118	1.000	1.000	0.118
0.036	0.119	1.000	1.000	0.119
0.04	0.122	1.000	1.000	0.122
0.042	0.123	1.000	1.000	0.123
0.044	0.125	1.000	1.000	0.125
0.045	0.126	1.000	1.000	0.126
0.046	0.127	1.000	1.000	0.127
0.048	0.129	1.000	1.000	0.129
0.05	0.130	1.000	1.000	0.130
0.055	0.136	1.000	1.000	0.136
0.06	0.142	1.000	1.000	0.142

0.065	0.148	1.000	1.000	0.148
0.067	0.150	1.000	1.000	0.150
0.07	0.154	1.000	1.000	0.154
0.075	0.160	1.000	1.000	0.160
0.08	0.166	1.000	1.000	0.166
0.085	0.172	1.000	1.000	0.172
0.09	0.178	1.000	1.000	0.178
0.095	0.184	1.000	1.000	0.184
0.1	0.190	1.000	1.000	0.190
0.11	0.200	1.000	1.000	0.200
0.12	0.210	1.000	1.000	0.210
0.13	0.218	1.000	1.000	0.218
0.133	0.220	1.000	1.000	0.220
0.14	0.225	1.000	1.000	0.225
0.15	0.232	1.000	1.000	0.232
0.16	0.236	1.000	1.000	0.236
0.17	0.239	1.000	1.000	0.239
0.18	0.242	1.000	1.000	0.242
0.19	0.245	1.000	1.000	0.245
0.2	0.246	1.000	1.000	0.246
0.22	0.246	1.000	1.000	0.246
0.24	0.244	1.000	1.000	0.244
0.25	0.243	1.000	1.000	0.243
0.26	0.242	1.000	1.000	0.242
0.28	0.239	1.000	1.000	0.239
0.29	0.238	1.000	1.000	0.238
0.3	0.236	1.000	1.000	0.236
0.32	0.231	1.000	1.000	0.231
0.34	0.225	1.000	1.000	0.225
0.35	0.223	1.000	1.000	0.223
0.36	0.220	1.000	1.000	0.220
0.38	0.215	1.000	1.000	0.215
0.4	0.210	1.000	1.000	0.210
0.42	0.204	1.000	1.000	0.204
0.44	0.199	1.000	1.000	0.199
0.45	0.197	1.000	1.000	0.197
0.46	0.194	1.000	1.000	0.194
0.48	0.189	1.000	1.000	0.189
0.5	0.185	1.000	1.000	0.185
0.55	0.171	1.000	1.000	0.171
0.6	0.160	1.000	1.000	0.160
0.65	0.150	1.000	1.000	0.150
0.667	0.146	1.000	1.000	0.146
0.7	0.141	1.000	1.000	0.141
0.75	0.133	1.000	1.000	0.133
0.8	0.124	1.000	1.000	0.124

0.85	0.117	1.000	1.000	0.117
0.9	0.110	1.000	1.000	0.110
0.95	0.104	1.000	1.000	0.104
1	0.099	1.000	1.000	0.099
1.1	0.088	1.000	1.000	0.088
1.2	0.079	1.000	1.000	0.079
1.3	0.071	1.000	1.000	0.071
1.4	0.064	1.000	1.000	0.064
1.5	0.058	1.000	1.000	0.058
1.6	0.053	1.000	1.000	0.053
1.7	0.048	1.000	1.000	0.048
1.8	0.044	1.000	1.000	0.044
1.9	0.040	1.000	1.000	0.040
2	0.037	1.000	1.000	0.037
2.2	0.032	1.000	1.000	0.032
2.4	0.028	1.000	1.000	0.028
2.5	0.026	1.000	1.000	0.026
2.6	0.025	1.000	1.000	0.025
2.8	0.022	1.000	1.000	0.022
3	0.020	1.000	1.000	0.020
3.2	0.018	1.000	1.000	0.018
3.4	0.017	1.000	1.000	0.017
3.5	0.016	1.000	1.000	0.016
3.6	0.016	1.000	1.000	0.016
3.8	0.015	1.000	1.000	0.015
4	0.014	1.000	1.000	0.014
4.2	0.013	1.000	1.000	0.013
4.4	0.012	1.000	1.000	0.012
4.6	0.011	1.000	1.000	0.011
4.8	0.011	1.000	1.000	0.011
5	0.010	1.000	1.000	0.010

San Andreas fault zone (North Coast section)

Fault ID:	308
Maximum Magnitude (MMax):	7.9
Fault Type:	RLSS
Fault Dip:	90 Deg
Dip Direction:	V
Bottom of Rupture Plane:	11.00 km
Top of Rupture Plane(Ztor):	0.00 km
Rrup	180.85 km
Rjb:	180.84 km
Rx:	180.85 km
Fnorm:	0
Frev:	0

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.039	1.000	1.000	0.039
0.02	0.039	1.000	1.000	0.039
0.022	0.039	1.000	1.000	0.039
0.025	0.039	1.000	1.000	0.039
0.029	0.040	1.000	1.000	0.040
0.03	0.040	1.000	1.000	0.040
0.032	0.040	1.000	1.000	0.040
0.035	0.040	1.000	1.000	0.040
0.036	0.040	1.000	1.000	0.040
0.04	0.041	1.000	1.000	0.041
0.042	0.041	1.000	1.000	0.041
0.044	0.041	1.000	1.000	0.041
0.045	0.042	1.000	1.000	0.042
0.046	0.042	1.000	1.000	0.042
0.048	0.042	1.000	1.000	0.042
0.05	0.042	1.000	1.000	0.042
0.055	0.043	1.000	1.000	0.043
0.06	0.044	1.000	1.000	0.044
0.065	0.045	1.000	1.000	0.045
0.067	0.045	1.000	1.000	0.045
0.07	0.046	1.000	1.000	0.046
0.075	0.047	1.000	1.000	0.047
0.08	0.048	1.000	1.000	0.048
0.085	0.050	1.000	1.000	0.050
0.09	0.051	1.000	1.000	0.051
0.095	0.052	1.000	1.000	0.052
0.1	0.054	1.000	1.000	0.054
0.11	0.057	1.000	1.000	0.057
0.12	0.061	1.000	1.000	0.061
0.13	0.064	1.000	1.000	0.064
0.133	0.065	1.000	1.000	0.065
0.14	0.067	1.000	1.000	0.067
0.15	0.070	1.000	1.000	0.070
0.16	0.073	1.000	1.000	0.073
0.17	0.076	1.000	1.000	0.076
0.18	0.079	1.000	1.000	0.079
0.19	0.082	1.000	1.000	0.082
0.2	0.085	1.000	1.000	0.085
0.22	0.088	1.000	1.000	0.088
0.24	0.092	1.000	1.000	0.092
0.25	0.093	1.000	1.000	0.093
0.26	0.095	1.000	1.000	0.095
0.28	0.097	1.000	1.000	0.097

0.29	0.098	1.000	1.000	0.098
0.3	0.098	1.000	1.000	0.098
0.32	0.098	1.000	1.000	0.098
0.34	0.098	1.000	1.000	0.098
0.35	0.098	1.000	1.000	0.098
0.36	0.098	1.000	1.000	0.098
0.38	0.097	1.000	1.000	0.097
0.4	0.096	1.000	1.000	0.096
0.42	0.096	1.000	1.000	0.096
0.44	0.096	1.000	1.000	0.096
0.45	0.096	1.000	1.000	0.096
0.46	0.096	1.000	1.000	0.096
0.48	0.096	1.000	1.000	0.096
0.5	0.096	1.000	1.000	0.096
0.55	0.093	1.000	1.000	0.093
0.6	0.091	1.000	1.000	0.091
0.65	0.089	1.000	1.000	0.089
0.667	0.088	1.000	1.000	0.088
0.7	0.087	1.000	1.000	0.087
0.75	0.085	1.000	1.000	0.085
0.8	0.083	1.000	1.000	0.083
0.85	0.081	1.000	1.000	0.081
0.9	0.078	1.000	1.000	0.078
0.95	0.076	1.000	1.000	0.076
1	0.074	1.000	1.000	0.074
1.1	0.070	1.000	1.000	0.070
1.2	0.067	1.000	1.000	0.067
1.3	0.063	1.000	1.000	0.063
1.4	0.060	1.000	1.000	0.060
1.5	0.057	1.000	1.000	0.057
1.6	0.054	1.000	1.000	0.054
1.7	0.051	1.000	1.000	0.051
1.8	0.049	1.000	1.000	0.049
1.9	0.047	1.000	1.000	0.047
2	0.045	1.000	1.000	0.045
2.2	0.040	1.000	1.000	0.040
2.4	0.037	1.000	1.000	0.037
2.5	0.035	1.000	1.000	0.035
2.6	0.034	1.000	1.000	0.034
2.8	0.031	1.000	1.000	0.031
3	0.029	1.000	1.000	0.029
3.2	0.027	1.000	1.000	0.027
3.4	0.026	1.000	1.000	0.026
3.5	0.025	1.000	1.000	0.025
3.6	0.024	1.000	1.000	0.024
3.8	0.023	1.000	1.000	0.023

4	0.021	1.000	1.000	0.021
4.2	0.020	1.000	1.000	0.020
4.4	0.019	1.000	1.000	0.019
4.6	0.018	1.000	1.000	0.018
4.8	0.018	1.000	1.000	0.018
5	0.017	1.000	1.000	0.017

PROBABILISTIC

Probabilistic Model
USGS Seismic Hazard Map(2008) 975 Year Return Period

Period	SA(Base Spectrum)	Basin Factor	Near Fault Factor(Not Applied)	SA(Final Spectrum)
0.01	0.206	1.000	1.000	0.206
0.02	0.241	1.000	1.000	0.241
0.022	0.246	1.000	1.000	0.246
0.025	0.254	1.000	1.000	0.254
0.029	0.262	1.000	1.000	0.262
0.03	0.265	1.000	1.000	0.265
0.032	0.268	1.000	1.000	0.268
0.035	0.274	1.000	1.000	0.274
0.036	0.276	1.000	1.000	0.276
0.04	0.282	1.000	1.000	0.282
0.042	0.286	1.000	1.000	0.286
0.044	0.289	1.000	1.000	0.289
0.045	0.290	1.000	1.000	0.290
0.046	0.292	1.000	1.000	0.292
0.048	0.294	1.000	1.000	0.294
0.05	0.297	1.000	1.000	0.297
0.055	0.304	1.000	1.000	0.304
0.06	0.310	1.000	1.000	0.310
0.065	0.315	1.000	1.000	0.315
0.067	0.318	1.000	1.000	0.318
0.07	0.321	1.000	1.000	0.321
0.075	0.326	1.000	1.000	0.326
0.08	0.331	1.000	1.000	0.331
0.085	0.335	1.000	1.000	0.335
0.09	0.340	1.000	1.000	0.340
0.095	0.344	1.000	1.000	0.344
0.1	0.348	1.000	1.000	0.348
0.11	0.360	1.000	1.000	0.360
0.12	0.372	1.000	1.000	0.372
0.13	0.383	1.000	1.000	0.383
0.133	0.386	1.000	1.000	0.386

0.14	0.394	1.000	1.000	0.394
0.15	0.404	1.000	1.000	0.404
0.16	0.413	1.000	1.000	0.413
0.17	0.423	1.000	1.000	0.423
0.18	0.432	1.000	1.000	0.432
0.19	0.440	1.000	1.000	0.440
0.2	0.449	1.000	1.000	0.449
0.22	0.444	1.000	1.000	0.444
0.24	0.440	1.000	1.000	0.440
0.25	0.437	1.000	1.000	0.437
0.26	0.436	1.000	1.000	0.436
0.28	0.432	1.000	1.000	0.432
0.29	0.430	1.000	1.000	0.430
0.3	0.428	1.000	1.000	0.428
0.32	0.423	1.000	1.000	0.423
0.34	0.417	1.000	1.000	0.417
0.35	0.415	1.000	1.000	0.415
0.36	0.412	1.000	1.000	0.412
0.38	0.408	1.000	1.000	0.408
0.4	0.403	1.000	1.000	0.403
0.42	0.399	1.000	1.000	0.399
0.44	0.395	1.000	1.000	0.395
0.45	0.393	1.000	1.000	0.393
0.46	0.392	1.000	1.000	0.392
0.48	0.388	1.000	1.000	0.388
0.5	0.385	1.000	1.000	0.385
0.55	0.366	1.000	1.020	0.366
0.6	0.350	1.000	1.040	0.350
0.65	0.336	1.000	1.060	0.336
0.667	0.331	1.000	1.067	0.331
0.7	0.323	1.000	1.080	0.323
0.75	0.312	1.000	1.100	0.312
0.8	0.299	1.000	1.120	0.299
0.85	0.287	1.000	1.140	0.287
0.9	0.276	1.000	1.160	0.276
0.95	0.267	1.000	1.180	0.267
1	0.258	1.000	1.200	0.258
1.1	0.239	1.000	1.200	0.239
1.2	0.223	1.000	1.200	0.223
1.3	0.209	1.000	1.200	0.209
1.4	0.197	1.000	1.200	0.197
1.5	0.186	1.000	1.200	0.186
1.6	0.177	1.000	1.200	0.177
1.7	0.168	1.000	1.200	0.168
1.8	0.161	1.000	1.200	0.161
1.9	0.154	1.000	1.200	0.154

2	0.148	1.000	1.200	0.148
2.2	0.132	1.000	1.200	0.132
2.4	0.119	1.000	1.200	0.119
2.5	0.113	1.000	1.200	0.113
2.6	0.108	1.000	1.200	0.108
2.8	0.099	1.000	1.200	0.099
3	0.091	1.000	1.200	0.091
3.2	0.084	1.000	1.200	0.084
3.4	0.077	1.000	1.200	0.077
3.5	0.074	1.000	1.200	0.074
3.6	0.071	1.000	1.200	0.071
3.8	0.066	1.000	1.200	0.066
4	0.062	1.000	1.200	0.062
4.2	0.059	1.000	1.200	0.059
4.4	0.056	1.000	1.200	0.056
4.6	0.053	1.000	1.200	0.053
4.8	0.050	1.000	1.200	0.050
5	0.048	1.000	1.200	0.048

MINIMUM DETERMINISTIC SPECTRUM

Period	SA
0.01	0.226
0.02	0.229
0.022	0.232
0.025	0.236
0.029	0.241
0.03	0.242
0.032	0.246
0.035	0.251
0.036	0.253
0.04	0.261
0.042	0.265
0.044	0.269
0.045	0.271
0.046	0.273
0.048	0.277
0.05	0.281
0.055	0.295
0.06	0.309
0.065	0.323
0.067	0.328
0.07	0.336
0.075	0.350

0.08	0.363
0.085	0.377
0.09	0.390
0.095	0.403
0.1	0.416
0.11	0.436
0.12	0.454
0.13	0.471
0.133	0.475
0.14	0.485
0.15	0.498
0.16	0.504
0.17	0.509
0.18	0.513
0.19	0.516
0.2	0.519
0.22	0.514
0.24	0.508
0.25	0.505
0.26	0.501
0.28	0.494
0.29	0.490
0.3	0.486
0.32	0.476
0.34	0.467
0.35	0.462
0.36	0.457
0.38	0.447
0.4	0.438
0.42	0.426
0.44	0.415
0.45	0.410
0.46	0.405
0.48	0.395
0.5	0.385
0.55	0.357
0.6	0.332
0.65	0.311
0.667	0.304
0.7	0.292
0.75	0.275
0.8	0.259
0.85	0.245
0.9	0.232
0.95	0.220
1	0.210

1.1	0.190
1.2	0.173
1.3	0.158
1.4	0.145
1.5	0.134
1.6	0.124
1.7	0.115
1.8	0.106
1.9	0.099
2	0.093
2.2	0.082
2.4	0.073
2.5	0.069
2.6	0.066
2.8	0.060
3	0.054
3.2	0.050
3.4	0.046
3.5	0.044
3.6	0.043
3.8	0.040
4	0.037
4.2	0.035
4.4	0.032
4.6	0.031
4.8	0.029
5	0.027

Envelope Data

Period	SA
0.01	0.226
0.02	0.241
0.022	0.246
0.025	0.254
0.029	0.262
0.03	0.265
0.032	0.268
0.035	0.274
0.036	0.276
0.04	0.282
0.042	0.286
0.044	0.289
0.045	0.290
0.046	0.292
0.048	0.294
0.05	0.297

0.055	0.304
0.06	0.310
0.065	0.323
0.067	0.328
0.07	0.336
0.075	0.350
0.08	0.363
0.085	0.377
0.09	0.390
0.095	0.403
0.1	0.416
0.11	0.436
0.12	0.454
0.13	0.471
0.133	0.475
0.14	0.485
0.15	0.498
0.16	0.504
0.17	0.509
0.18	0.513
0.19	0.516
0.2	0.519
0.22	0.514
0.24	0.508
0.25	0.505
0.26	0.501
0.28	0.494
0.29	0.490
0.3	0.486
0.32	0.476
0.34	0.467
0.35	0.462
0.36	0.457
0.38	0.447
0.4	0.438
0.42	0.426
0.44	0.415
0.45	0.410
0.46	0.405
0.48	0.395
0.5	0.385
0.55	0.366
0.6	0.350
0.65	0.336
0.667	0.331
0.7	0.323

0.75	0.312
0.8	0.299
0.85	0.287
0.9	0.276
0.95	0.267
1	0.258
1.1	0.239
1.2	0.223
1.3	0.209
1.4	0.197
1.5	0.186
1.6	0.177
1.7	0.168
1.8	0.161
1.9	0.154
2	0.148
2.2	0.132
2.4	0.119
2.5	0.113
2.6	0.108
2.8	0.099
3	0.091
3.2	0.084
3.4	0.077
3.5	0.074
3.6	0.071
3.8	0.066
4	0.062
4.2	0.059
4.4	0.056
4.6	0.053
4.8	0.050
5	0.048

APPENDIX D

Abutments: Class 90 Pile Calculations

Abutment Pile Settlement Calculation

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1202~1.1SR\ENGINE~1\DRIVEN\DHSABUT.DVN
Project Name: Dead Horse Abutments Project Date: 07/28/2010
Project Client: MTCo
Computed By: WEN
Project Manager: WEN

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 ft
Length of Square Side: 12.00 in

ULTIMATE CONSIDERATIONS

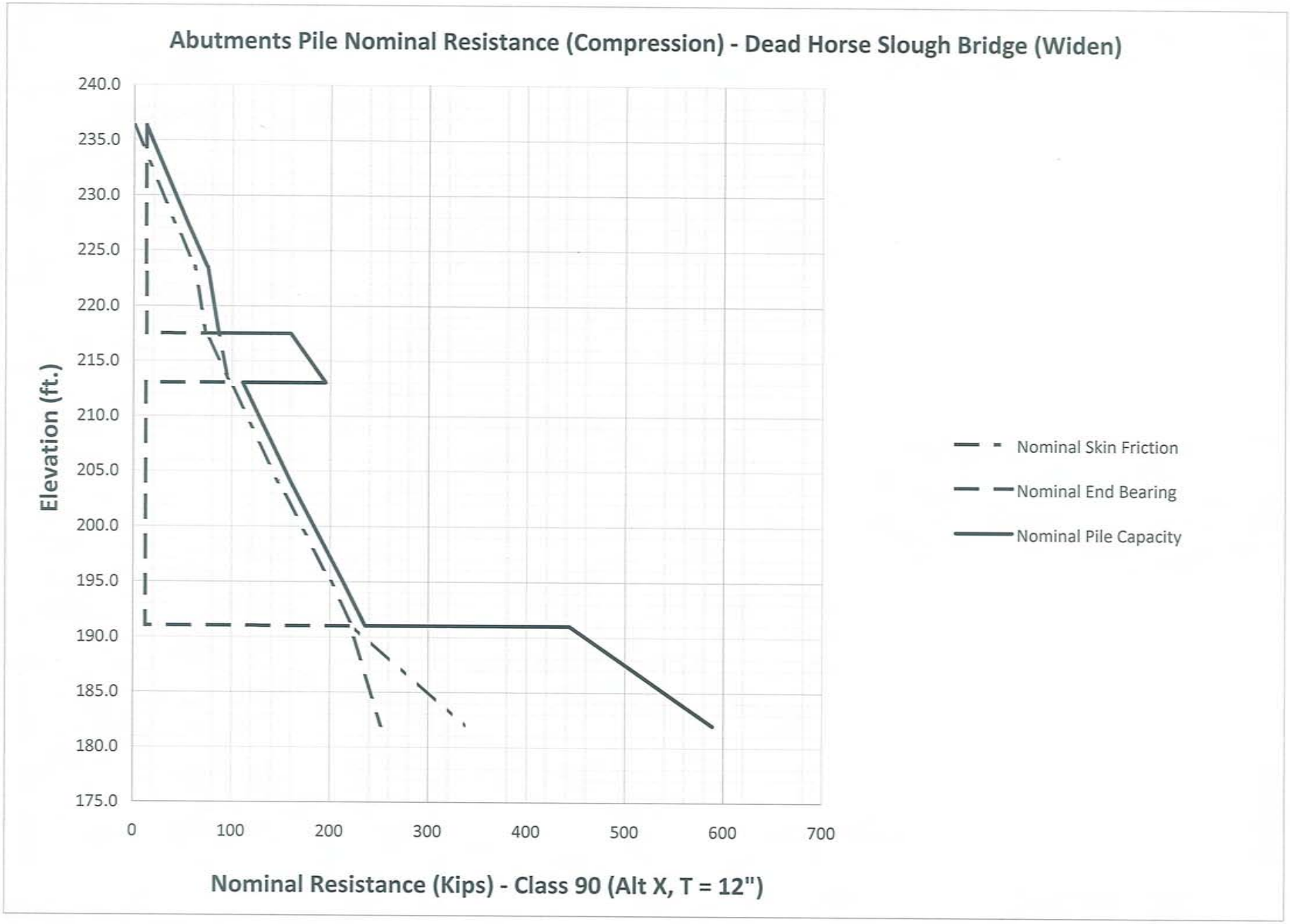
Water Table Depth At Time Of:	- Drilling:	9.90 ft
	- Driving/Restrike	9.90 ft
	- Ultimate:	9.90 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	12.80 ft	0.00%	119.00 pcf	1400.00 psf	T-79 Concrete
2	Cohesionless	6.00 ft	0.00%	106.00 pcf	27.0/27.0	Nordlund
3	Cohesionless	4.50 ft	0.00%	107.00 pcf	36.0/36.0	Nordlund
4	Cohesive	22.00 ft	0.00%	106.00 pcf	1400.00 psf	T-79 Concrete
5	Cohesionless	29.00 ft	0.00%	107.00 pcf	38.0/38.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.05 Kips	12.60 Kips	12.65 Kips
9.01 ft	42.89 Kips	12.60 Kips	55.49 Kips
12.79 ft	61.97 Kips	12.60 Kips	74.57 Kips
12.81 ft	62.04 Kips	13.32 Kips	75.36 Kips
18.79 ft	73.29 Kips	13.32 Kips	86.61 Kips
18.81 ft	73.36 Kips	86.31 Kips	159.67 Kips
23.29 ft	98.35 Kips	96.91 Kips	195.26 Kips
23.31 ft	98.46 Kips	12.60 Kips	111.06 Kips
32.31 ft	147.46 Kips	12.60 Kips	160.06 Kips
41.31 ft	200.71 Kips	12.60 Kips	213.31 Kips
45.29 ft	223.31 Kips	12.60 Kips	235.91 Kips
45.31 ft	223.49 Kips	220.33 Kips	443.82 Kips
54.31 ft	336.59 Kips	252.33 Kips	588.92 Kips
63.31 ft	465.01 Kips	268.60 Kips	733.61 Kips
72.31 ft	608.74 Kips	268.60 Kips	877.34 Kips
74.29 ft	642.41 Kips	268.60 Kips	911.01 Kips



Dead Horse Slough Bridge (Widen)
Bridge No. 12-0135
03-BUT-32, PM 11.08, EA:1E4901

Dead Horse Slough Widen
BCI No. 1202.1
08/03/10
By: WEN

Abutment Pile Settlement Calculations: Class 90 Piles

(Foundation Analysis and Design, Bowles, 5th edition, 1996)

Axial Pile Compression

Service-I Limit Per Pile Load (lbs)		75000
A	*Average Axial Load (lbs)	37500
B	Pile Length (in.)	438
C	Tip Area (sq. in.)	144
D	Concrete Modulus of Elasticity (psi)	4415201
Axial Compression (in.)		**0.03**

*Allowable Capacity Reduced by 50% Due to Skin Friction

Axial Compression = (A x B)/(C x D)

Point Settlement

A	Point Bearing Pressure (psi)	520.8
B	Pile Diameter (in.)	12
C	Poisson's Ratio	0.3
D	Point Soil Stress-Strain Modulus (psi)	4800
E	Shape Factor	1
F	Fox Embeddment Factor	0.5
G	Reduction Factor for Skin Friction	0.5
Point Settlement (in.)		**0.30**

A = Allowable Pile Capacity x Tip Area

F = 0.55 if L/D >/= to 5, 0.5 if greater than 5

Point Settlement = A x {B x (1-C^2)/D} x E x F x G

Total Pile Settlement = 0.32 in.
8.2 mm

APPENDIX E

Piers: 15-inch PC/PS Concrete Pile Extension Calculations

Pier Pile Settlement Calculation

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1202~1.1SR\ENGINE~1\DRIVEN\BNTEND.DVN
Project Name: Dead Horse END
Project Date: 09/20/2010
Project Client: MTCo
Computed By: WEN
Project Manager: WEN

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 ft
Length of Square Side: 13.30 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	5.60 ft
	- Driving/Restrike	5.60 ft
	- Ultimate:	5.60 ft
Ultimate Considerations:	- Local Scour:	7.00 ft
	- Long Term Scour:	0.30 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	8.50 ft	0.00%	119.00 pcf	1400.00 psf	T-79 Concrete
2	Cohesionless	6.00 ft	0.00%	106.00 pcf	27.0/27.0	Nordlund
3	Cohesionless	4.50 ft	0.00%	107.00 pcf	36.0/36.0	Nordlund
4	Cohesive	22.00 ft	0.00%	106.00 pcf	1400.00 psf	T-79 Concrete
5	Cohesionless	29.00 ft	0.00%	107.00 pcf	38.0/38.0	Nordlund

DRIVING - SUMMARY OF CAPACITIES

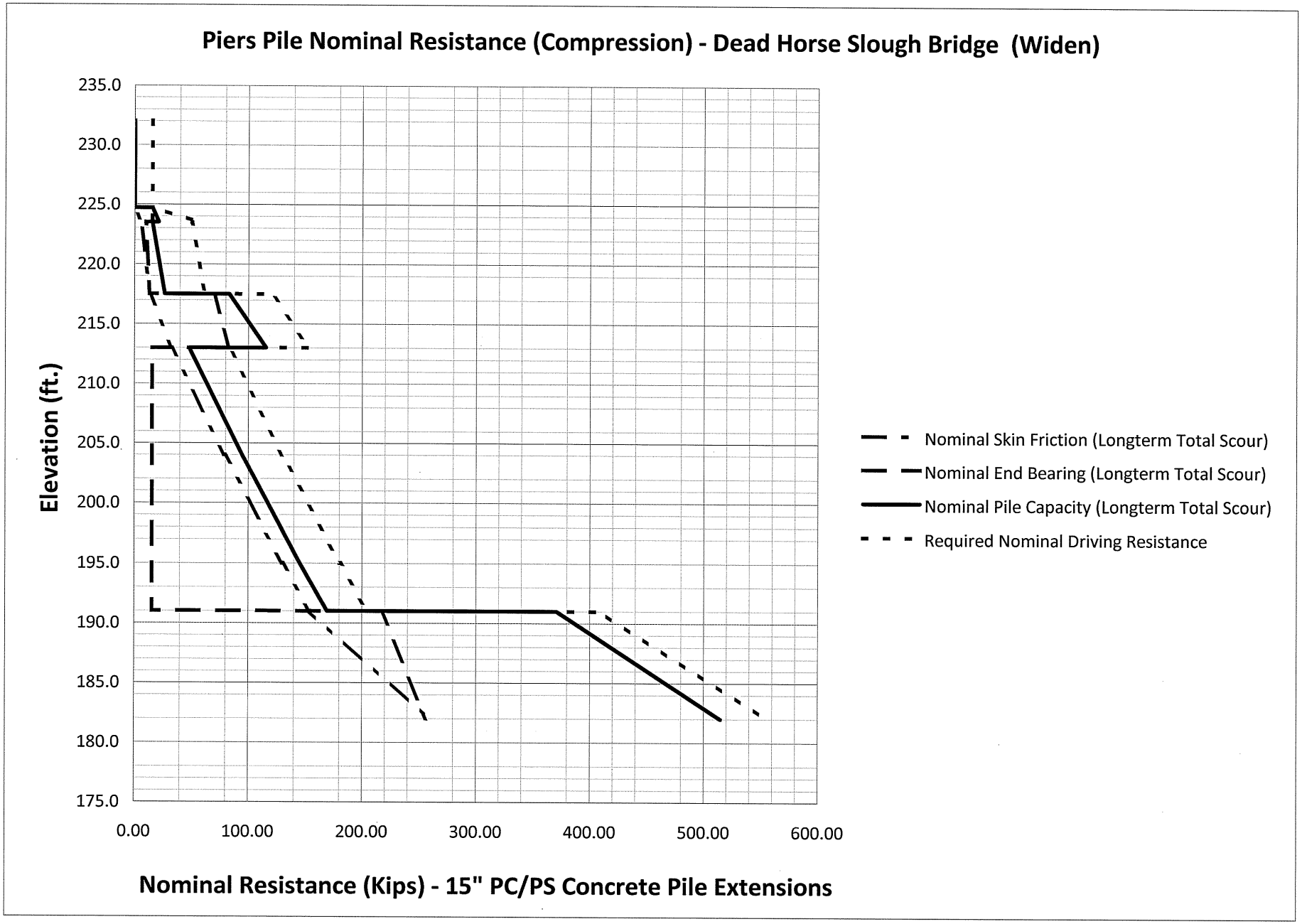
Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.05 Kips	15.48 Kips	15.53 Kips
8.49 ft	44.79 Kips	15.48 Kips	60.27 Kips
8.51 ft	44.86 Kips	10.39 Kips	55.24 Kips
14.49 ft	53.88 Kips	13.65 Kips	67.53 Kips
14.51 ft	53.95 Kips	72.21 Kips	126.16 Kips
18.99 ft	76.05 Kips	85.42 Kips	161.46 Kips
19.01 ft	76.16 Kips	15.48 Kips	91.63 Kips
28.01 ft	128.31 Kips	15.48 Kips	143.79 Kips
37.01 ft	185.44 Kips	15.48 Kips	200.91 Kips
40.99 ft	212.28 Kips	15.48 Kips	227.76 Kips
41.01 ft	212.47 Kips	220.55 Kips	433.02 Kips
50.01 ft	331.58 Kips	259.85 Kips	591.43 Kips
59.01 ft	470.17 Kips	299.16 Kips	769.33 Kips
68.01 ft	628.26 Kips	329.95 Kips	958.21 Kips
69.99 ft	665.65 Kips	329.95 Kips	995.60 Kips

x .89

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
7.29 ft	0.00 Kips	0.00 Kips	0.00 Kips
7.30 ft	0.00 Kips	15.48 Kips	15.48 Kips
8.49 ft	6.28 Kips	15.48 Kips	21.76 Kips
8.51 ft	6.34 Kips	9.94 Kips	16.29 Kips
14.49 ft	15.04 Kips	13.20 Kips	28.24 Kips
14.51 ft	15.10 Kips	69.85 Kips	84.95 Kips
18.99 ft	36.54 Kips	83.06 Kips	119.59 Kips
19.01 ft	36.64 Kips	15.48 Kips	52.12 Kips
28.01 ft	88.80 Kips	15.48 Kips	104.28 Kips
37.01 ft	145.92 Kips	15.48 Kips	161.40 Kips
40.99 ft	172.77 Kips	15.48 Kips	188.25 Kips
41.01 ft	172.96 Kips	217.06 Kips	390.01 Kips
50.01 ft	290.33 Kips	256.36 Kips	546.69 Kips
59.01 ft	427.19 Kips	295.66 Kips	722.85 Kips
68.01 ft	583.54 Kips	329.95 Kips	913.49 Kips
69.99 ft	620.56 Kips	329.95 Kips	950.50 Kips

x .89



Dead Horse Slough Bridge (Widen)
Bridge No. 12-0135
03-BUT-32, PM 11.08, EA:1E4901

Dead Horse Slough Widen
BCI No. 1202.1
08/03/10
By: WEN

Pier Pile Settlement Calculations: 15" PC/PS Concrete Pile Extensions

(Foundation Analysis and Design, Bowles, 5th edition, 1996)

Axial Pile Compression

Service-I Limit Per Pile Load (lbs)		100000
A	*Average Axial Load (lbs)	50000
B	Pile Length (in.)	444
C	Tip Area (sq. in.)	177
D	Concrete Modulus of Elasticity (psi)	4415201
Axial Compression (in.)		**0.028**

*Allowable Capacity Reduced by 50% Due to Skin Friction

Axial Compression = (A x B)/(C x D)

Point Settlement

A	Point Bearing Pressure (psi)	565.0
B	Pile Diameter (in.)	15
C	Poisson's Ratio	0.3
D	Point Soil Stress-Strain Modulus (psi)	8000
E	Shape Factor	1
F	Fox Embeddment Factor	0.5
G	Reduction Factor for Skin Friction	0.5
Point Settlement (in.)		**0.241**

A = Allowable Pile Capacity x Tip Area

F = 0.55 if L/D >/= to 5, 0.5 if greater than 5

Point Settlement = A x {B x (1-C^2)/D} x E x F x G

Total Pile Settlement = 0.27 in.
6.8 mm

APPENDIX F

Caltrans Review comments and BCI Response

Office of Geotechnical Engineering
Comment & Response Form

(Revised 10/21/09)

General Project Information (OSFP Liaison to complete)	Review Phase (OSFP Liaison to complete)	Reviewer Information (Reviewer Liaison to complete)
Dist: 03 **EA:** 1E4901 (E-Fis 0300000264) **Project Name:** Dead Horse Slough (Widen) **Senior Eng.:** Mark Desalvatore **Phone:** 916-227-5391 **e-mail:** *mark_desalvatore@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. 1) _ Construction Other:	**Reviewer Name:** Fernando De Haro **Functional Unit:** 3659 **Cost Center:** 059 **Phone Number:** 916-227-4556 **e-mail:** *Fernando_de_haro@dot.ca.gov* **Date of Review:** 9/7/2010 **Structure Name*:** Dead Horse Slough (Widen) **Bridge No*:** 12-0135 (*Use if necessary to when comment sheets are by individual structure)

Consultant Information (to be filled in by Consultant)

Consulting Company Lead (First and Last Name)	Consulting Company Name	Phone Number	e-mail	Response Date
Eric Nichols	Blackburn Consulting	(916) 375 8706	ericn@blackburnconsulting.com	9/21/2010

#	Doc./ Page (See Note 1)	Section	Reviewer Comments	Consultant Responses	✓
1	P-6 of 18	Pier Layout	Section C-C states "Octagonal or Round section to match existing". Existing columns are round. No need to say "Octagonal".	This comment to be addressed by Mark Thomas & Company, Inc.	
2	P-13 of 18	LOTB 1 of 6	Borehole size in A-10-B2 does not match the borehole size in Sheet 14 of 18. Make proper correction.	BCI corrected the borehole size.	
3	P-14 of 18	LOTB 2 of 6	Rock Classification is missing "Rock Identification". Add the rock identification after the rock family name (for example: Sedimentary Rock (Shale)).	BCI has edited the rock descriptions and included the rock identification after the rock family name.	
4	P-14 of 18	LOTB 2 of 6	Rock identification and descriptors are repeated frequently. Create one rock layer and add the new descriptor information as it varies through out the layer. Core run numbers don't need to be shown. Same as in soil layers, sample number doesn't need to be shown.	BCI has edited the rock descriptions. We include core run numbers and/or soil sample numbers for cross reference with laboratory test results.	

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

✓= Comment Resolved (for Reviewer's use)

5	P-14 of 18	LOTB 2 of 6	Rock descriptors such as weathering and hardness range from the lowest to the highest (for example, extremely hard and very soft or slightly weathered to fresh and decomposed). Review the descriptors using as reference the Note section in Figure 2-29. If decomposed, soil classification shall be included.	BCI has edited the rock descriptions. We include soil classification where rock is described as very intensely weathered or decomposed (consistent with the Caltrans 2007 Soil and Rock Logging Manual).	
6	P-14 of 18	LOTB 2 of 6	Note 4 is not in accordance with the 2007 Soil Logging Manual. Remove it.	BCI removed Note 4.	
7	P-14 of 18	LOTB 2 of 6	Soil Borings A-10-B1 and A-10-B2 present blowcounts **50/0.5** For test penetration less than 6 inches shall be presented as refusal. See Appendix A.8.	BCI edited the blowcount for test penetration less than 6 inches on the LOTB.	
8	P-15 of 18	LOTB 3 of 6	Rotary Boring Detail is not as per 2007 Manual. See Section 5.2.5.2. Make proper corrections.	BCI has made edits to the Rotary Boring Detail so that it conforms to the Caltrans 2007 Soil and Rock Logging Manual.	
10	FR-2	Section 2.1-Caltrans	First and last sentences - "The three cone penetrometer borings". Add "dynamic" before "cone". Do the same in Section 4.3.2.	BCI has edited the text of the report to include “dynamic.”	
11	FR-14	Section – 9.5.1 Compressive Resistance	1st Paragraph – It mentions that pile resistance is about 25% end bearing and 75% skin friction. At tip elevation, the end bearing layer will be in very dense gravel or rock. Based on design calculations, end bearing is grater than skin friction. Check and clarify these numbers.	BCI has edited the text of the report to clarify that the nominal compressive resistance of the piles will essentially be obtained through end bearing.	
12	FR-14	Section – 9.5.1 Compressive Resistance and Appendix E	3rd Paragraph – It mentions that pile square sizes of 11.8 inches and 13.3 inches were used for skin friction and end bearing contributions, respectively. Design pile calculations for 11.8 inches pile size underestimates soil volume displacement. Therefore, it underestimates K_s. **Option 1** - A simplistic approach is to reduce the skin friction values obtained in pile size 13.3 inches. Affect skin resistance values by a factor of 0.89 (11.8/13.3). Then, adjust skin resistance for drilling to assist to elevation 213ft. **Option 2** - Pile tip elevation is at bedrock or very dense gravel. Design pile for end bearing only.	BCI conservatively bases the pile tip elevations on end bearing contribution only (Option 2).	
13	FR-16	Section - 11.2 Abutment and Pier Piles	Address pre-drilling on new abutments.	BCI now includes reference to Section 49-1.06 of the Standard Specifications regarding pre-drilling at abutments.	
14	FR-16	Section - 11.2 Abutment and Pier Piles	Last paragraph - Engineering News Formula is not used to determine minimum blowcount. Remove sentence.	BCI has removed reference to the Engineering News Formula.	
15	FR-17	Section - 11.2 Abutment and Pier Piles	1st Paragraph – It mentions pile acceptance criteria within 5 feet of tip elevation. It may be possible to reach high blowcount above 5ft from tip elevation. Include acceptance criteria for pile maximum capacity when reaching refusal above tip elevation.	In case of high blowcount above 5ft from pile tip elevation, contact BCI to review pile driving records and construction conditions for additional pile acceptance criteria if needed.	