FINAL FOUNDATION REPORT

Dover Canyon Road at Jack Creek Bridge
Existing Br. No. 49C0037
New Br. No. 49C0472
San Luis Obispo County, California

Prepared By:

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January 30, 2020

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January 30, 2020
CAInc File No. 17-375.1

Julie Passalacqua, PE
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701 University Avenue, Suite 200
Sacramento, CA 95825

Subject: **FINAL FOUNDATION REPORT**
Dover Canyon Road at Jack Creek Bridge
Existing Br. No. 49C0037
New Br. No. 49C0472
San Luis Obispo County, California

Dear Ms. Passalacqua,

Crawford & Associates, Inc. (CAInc) prepared this Final Foundation Report for the Replacement of Dover Canyon Road at Jack Creek Bridge Project in San Luis Obispo County, California. CAInc prepared this report in accordance with our May 5, 2017 agreement between CAInc and Mark Thomas.

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

Sincerely,

Crawford & Associates, Inc.,

Reviewed by,

Hailey Wagenman

Hailey F. Wagenman
Project Engineer

Benjamin D. Crawford
Principal

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TABLE OF CONTENTS

LIST OF TABLES

FIGURES

APPENDIX I

APPENDIX II

APPENDIX III

1 INTRODUCTION

1.1 PURPOSE

Crawford & Associates, Inc. (CAInc), prepared this Final Foundation Report for the Dover Canyon Road at Jack Creek Bridge Replacement project located in San Luis Obispo County, California. This report provides geotechnical recommendations/considerations for the proposed new bridge foundations and approaches.

1.2 GEOTECHNICAL SERVICES

To prepare this report, CAInc:

- discussed the project with Ms. Julie Passalacqua and Mr. Victor Sherby at Mark Thomas (MT),
- discussed scour potential with Cathy Avila at Avila and Associates Consulting Engineers, Inc.,
- reviewed published geologic maps and literature pertaining to the site,
- reviewed the "General Plan" prepared by MT, received January 29, 2020,
- completed a Preliminary Foundation Report dated May 2, 2018,
- completed four exploratory borings to a maximum depth of 35 ft on May 7-8, 2018,
- completed four dynamic cone penetrometer tests to a maximum depth of about 15.9 ft on June 12, 2018,
- reviewed the Design Hydraulic Study Report completed by Avila and Associates Consulting Engineers, Inc., dated January 17, 2020,
- completed laboratory testing on soil/rock samples obtained during the subsurface exploration, and
- performed engineering evaluation and analysis for new bridge foundation design.

1.3 PROJECT DATUM

All elevations referenced in this report are based on the NGVD 88 Vertical Datum and NAD 83 Horizontal Datum.

2 PROJECT DESCRIPTION

2.1 PROJECT LOCATION

The project site is located about 1.2 miles southwest of Vineyard Drive and Dover Canyon Road intersection in San Luis Obispo County, California. Site coordinates are approximately latitude 35.577814° N and longitude -120.835061° W. The site location is shown on the attached Figure 1.

2.2 SITE DESCRIPTION

The existing bridge crosses Jack Creek, which flows south-easterly within a 60±foot (ft) wide channel. The existing bridge, built in 1920, is about 16-ft wide by 63-ft long is on about a 15 degree skewed alignment to the creek. The existing bridge deck is approximately 18 feet above the channel bottom. The bridge is a single span, steel pony truss with a timber deck supported on reinforced concrete abutments on an unknown foundation type with flared wingwalls. Currently, the approach roads unpaved.

No rock outcrop was observed within the channel banks or along the northeastern road cuts, however it appears that the existing southwestern abutment sits on top of a rock outcrop. Also, a large boulder or rock outcrop (approximately 5 to 6 ft in length) was observed within the center of the channel. Smaller, localized boulders (approximately 2 to 3 ft in length) were observed along the channel banks.

The Caltrans Bridge Inspection Report dated March 5, 2012 notes that there is distress along the top chord at the left truss, and surficial “freckled and light” rust throughout all the steel elements of the structure. Additionally, the report states that there is a concrete piece missing below the left bearing of Abutment 1, however the remaining concrete is in relatively good condition.

The bridge site is located in a rural area and land use near the bridge site is generally undeveloped, open land. The nearest structure to the site is a residence located about 0.40 miles northwest of the bridge.

Existing overhead utilities are located along the southeastern edge of the bridge deck. An underground AT&T fiber optic cable is located along the south eastern shoulder on either side of the bridge. The AT&T line attaches to the overhead lines when it crosses the bridge.

2.3 PROJECT INFORMATION

It is planned to replace the existing bridge with a slightly longer, wider clear-span bridge on a similar alignment/skew as the existing bridge. The preliminary General Plan (received January 29, 2020) shows the proposed structure as a single-span, precast/prestressed concrete voided slab bridge. The new bridge will be 79 ft long and 26 ft wide. The substructure is shown as concrete wall, seat-type abutments with cantilevered wingwalls. The new bridge deck grade is shown on a slight vertical curve that passes through elev. 1084.65 at Abutment-1 (west, Begin Bridge Sta. 19+70.00) and elev. 1085.44 at Abutment-2 (east, End Bridge Sta. 20+49.00).

The plans show placement of Rock Slope Protection (RSP) at 1.5H:1V (horizontal:vertical distance) slope in front of the abutments and slightly upstream/downstream from the bridge. No other channel improvements are indicated for this project.

New approach roadway improvements will also be completed as part of the project on each end of the proposed bridge. New/widened roadway sections will be raised (about 2±ft) to accommodate increased bridge deck grade. No separate retaining walls are proposed as part of this project.

3 EXCEPTIONS TO POLICIES AND PROCEDURES

At the date of this report, we understand that there are no exceptions to Departmental policies and procedures for this project.

4 FIELD INVESTIGATION AND FIELD TESTING PROGRAM

4.1 EXPLORATORY TEST BORINGS

CAInc retained Moore and Twining to drill and sample a total of four test borings on May 7-8, 2018. Boring depths ranged from 11.0 to 35.0 feet below ground surface (bgs).

Moore and Twining used a CME 75 truck mounted drill rig to complete the borings with 4.5-inch hollow-stem auger drilling equipment. Soil samples were recovered by means of a 2.0-inch O.D. "Standard Penetration" (SPT) split-spoon sampler with liners and a 3.0-inch O.D. "Standard California" spilt-spoon sampler with liners. Both samplers were advanced with standard 350 ft-lb striking force using an auto-hammer. An energy hammer analysis was not performed specific to this project/site; however, Moore and Twining reports an efficiency of 91%. The field recorded (uncorrected) blow counts are shown on the "Log of Test Borings" (LOTB) drawing attached as Appendix I. HQ diamond bit coring equipment was also used to advance borings A-18-001 and A-18-003, and to recover rock core samples.

CAInc retained Taber Drilling to drill, sample and rock core two additional borings on August 1, 2018. Boring depths ranged from 20.5 to 68 feet bgs.

Taber Drilling used a CME 55 truck mounted drill rig to complete the borings with 4-inch solid-stem auger drilling equipment as well as HQ diamond bit coring equipment used to advance boring RC-18-005 and obtain core samples. Soil samples were recovered by means of a 3.0-inch O.D. "Standard California" spilt-spoon sampler with liners. The sampler was advanced with standard 350 ft-lb striking force using an auto-hammer. An energy hammer analysis was not performed specific to this project/site; however ,Taber Drilling reports an efficiency of 79%. The field recorded (uncorrected) blow counts are shown on the "Log of Test Borings" (LOTB) drawing attached as Appendix I.

CAInc's Project Engineer, Hailey Wagenman, logged the test borings consistent with the Unified Soil Classification System (USCS) and the Caltrans 2010 Logging Manual. Selected portions of recovered soil drive samples were retained in sealed containers for laboratory testing and reference. Bulk soil samples were retained in sealed bags for laboratory testing and reference. Groundwater was not encountered within the auger portions of our borings completed between May 7 and 8, 2018 (lowest elevation of about 1049 ft); however, groundwater was encountered at about 18 feet bgs (elevation 1067 ft) in borings RC-18-005 and A-18-006 completed August 1, 2018. At completion, test borings were backfilled with lean cement grout per the county boring permit requirements.

The boring locations were measured in the field with respect to existing site features and then referenced to project stationing. The boring elevations are referenced to project datum provided by MT. The details and locations of test borings are shown on the "Log of Test Borings" (LOTB) drawing, provided in the Appendix I.

4.2 DYNAMIC CONE PENETRATION TESTS

The sampled test borings were supplemented with four Dynamic Cone Penetration (DCP) tests. A manually-operated Wildcat Dynamic Cone Penetrometer manufactured by Triggs Technology, Inc. was used to complete the DCP tests. The test consists of continuously driving a 1.4" O.D.

steel cone tip attached to a lead rod until effective refusal (50 blows per approximately 4-inches) is recorded. The rods are advanced using a hand-actuated, 35-lb safety drop hammer falling a distance of 15-inches. The DCP test provides an approximate quantification of a materials apparent density or stiffness.

CAInc referenced the DCP test locations to existing site features in the field, then to project stationing. The DCP results are provided in Appendix II, and the locations are shown on the attached Figure 2 and the LOTB. The DCPs were completed by CAInc's Project Engineers, Hailey Wagenman, Amando Castro and Kevin Escobedo on June 12, 2018.

5 LABORATORY TESTING PROGRAM

The following laboratory tests were completed on representative soil/rock samples obtained from the exploratory borings:

- Moisture Content - Dry Density (ASTM D2216 / D2937)
- Particle Size Analysis (ASTM D2487)
- Atterberg Limits (ASTM D4318)
- Unconfined Compression (ASTM D2166)
- Point Load Strength Test (ASTM D5731)

Laboratory tests were used for soil classification and strength parameters. Laboratory test results are shown in Appendix III.

6 SITE GEOLOGY AND SUBSURFACE CONDITIONS

6.1 SITE GEOLOGY

Published geologic mapping of the York Mountain Quadrangle[1] shows surficial materials at the site as Quaternary age Surficial Sediments (Qa) that generally consist of unconsolidated alluvial sands and gravels. The surrounding mountains are generally mapped as Marine Clastic Sedimentary Rocks (Kas, Cretaceous age) which includes light brown sandstone. Additionally, the Franciscan Assemblage (fg, Jurassic-Cretaceous age) and Marine Clastic Sedimentary (Kash, Cretaceous age) geologic formations, which consist of greenstone and dark gray shale, respectively, are present in local areas north of the project site.

Refer to the attached Figure 3 for a geologic map of the site vicinity.

6.2 SUBSURFACE CONDITIONS

Earth materials encountered in the borings are considered generally consistent with the published mapping. We identify two units in the borings that are considered significant to the proposed project, as summarized below.

Unit 1: This unit consists of loose to medium dense sand and decomposed sedimentary rock, and very stiff to hard clays with varying amounts of sand. The loose to medium dense sands

[1]Dibblee, T.W., and Minch, J.A., 2006, Geologic map of the York Mountain quadrangle, San Luis Obispo County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-217, scale 1:24,000

and decomposed sedimentary rock is generally closer to the abutment locations, and the very stiff to hard clays are primarily encountered on the northeastern approach. Based on our DCP results, we interpret Unit 1 to extend to a depth of about 16 feet (elev 1068±) at Abutment 1, and to about 11 feet (elev. 1073±) at Abutment 2.

Unit 2: Unit 2 consists of intensely to moderately weathered sedimentary rock (sandstone and/or shale). Unit 2 was encountered below Unit 1 and to the maximum depth explored (35± ft; lowest elev. 1049±). Uniaxial Compressive Strengths of tested sedimentary rock (shale) cores vary from 400 psi to 1,200 psi.

7 GROUNDWATER

Groundwater was not encountered in the auger portions of our borings (lowest elevation of about 1049 ft). In general, we expect:

- groundwater level in the vicinity of the bridge will be coincident with the surface water level in the creek with seasonal fluctuation of groundwater level within soils overlying the rock,
- overburden soils and upper portions of decomposed and very intensely weathered rock to be seasonally saturated,
- shallow ground water and seepage along the soil/rock interface during the winter months or extended periods of rainfall, and
- groundwater within the underlying less-weathered rock to be discontinuous, likely transmitted as seepage through rock discontinuities (e.g., fractures, joints, etc.).

Groundwater levels will fluctuate due to changes in the Jack Creek water level, precipitation, seasonal fluctuations, irrigation, pumping in nearby wells, and other factors.

8 AS-BUILT FOUNDATION DATA

No as-built foundation data was available to review at the time this report was prepared.

9 SCOUR EVALUATION

Based on the DCP results we obtained in the field, scour resistant rock was generally encountered at about 16 feet bgs (1068 ft elev.) at Abutment 1, and 11 feet bgs (1073 ft elev.) for Abutment 2. Scour data presented in the Design Hydraulic Study Report by Avila and Associates Consulting Engineers, Inc., dated January 17, 2020 is presented in Table 1. A thalweg elevation of 1064 ft is assumed.

Table 1: Scour Data

Support No.	Long Term (Degradation & Contraction) Scour Elev. (ft)	Short Term (Local) Scour Depth (ft)
Abut 1	1068	16
Abut 2	1073	16

Rock that is scour resistant is not scour proof. Some rock scour would be expected to occur over the life of the bridge, but the amount of scour would likely not affect bridge foundation elements established within the rock.

10 CORROSION EVALUATION

Based on the test results summarized below and current Caltrans guidelines, the site is not corrosive to structural concrete/steel foundation elements. The test results are only an indicator of soil corrosivity. Section 12 of Caltrans' March 2018 Corrosion Guidelines (Version 3.0) provides information regarding corrosion mitigation measures for structural elements and lists additional Caltrans guideline documents regarding corrosion mitigation. The designer should consult with a corrosion engineer if the test result values are considered significant.

Table 2 summarizes the soil corrosivity test results performed on soil samples obtained during our subsurface exploration.

Table 2: Soil Corrosion Test Summary

Boring / Sample No.	Depth (ft)	Elevation (ft)	pH	Minimum Resistivity (ohm-cm)	Chloride Content (ppm)	Sulfate Content (ppm)
A-18-003-1A	3.5 to 4.0	1080.5 to 1081.0	5.65	2,280	1.5	5.9
A-18-003-4A	16.0 to 16.5	1068.0 to 1068.5	6.77	620	18.6	178.1

For structural elements, Caltrans defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE wall design, Caltrans does not include minimum resistivity as a parameter to define a corrosive area for structures. Soil and water are not required to be tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm. (Caltrans Corrosion Guidelines Version 3.0, March 2018).

11 SEISMIC DESIGN INFORMATION AND RECOMMENDATIONS

11.1 GROUND MOTION

The Caltrans ARS Online (v2.3.09)[2] web-based tool was used to calculate both deterministic and probabilistic acceleration response spectra for the site based on criteria outlined in Appendix B of the April 2013 Caltrans Seismic Design Criteria (SDC) Version 1.7. The United States Geological Survey (USGS) Earthquake Hazards Program web-based tool was also used to calculate the probabilistic Mean/Modal Magnitude and Mean/Modal Site-to-Fault Distance Rupture for the site.

For our evaluation, we used latitude 35.577814° N and longitude 120.835061° W for the site coordinates and an estimated shear wave velocity (V_{S30}) of 360 meters per second (about 1,837 feet per second) that corresponds to a "very dense soil and soft rock" with 350 m/s < Vs < 750 m/s (Soil Profile Type C) for the upper 100 feet of the soil/rock profile.

[2] http://dap3.dot.ca.gov/ARS_Online/index.php, accessed January 17, 2020.

The V_{S30} value was determined for this site based on the soil data obtained from CAInc's 2018 soil exploration and correlations with SPT blow count N-values corrected for hammer efficiency using the equations outlined in Appendix A of Caltrans' *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations*, November 2012.

11.1.1 DETERMINISTIC EVALUATION

Based on Caltrans ARS Online (v2.3.09) and the 2012 Caltrans Fault Database v2b, the nearest deterministic seismic sources are the Oceanic – West Huasna fault, Rinconada 2011 CFM fault, and Hosgri fault, assigned the parameters shown in Table 3.

Table 3: Fault Data

Fault Parameters	Oceanic – West Huasna	Rinconada 2011 CFM	Hosgri
Fault Identification Number (FID)	223	209	213
Maximum Moment Magnitude (M_{max})	6.9	7.4	7.3
Fault Type	Reverse	Strike Slip	Strike Slip
Fault Dip (degrees)	58°	82°	80°
Dip Direction	Southwest	West	East
Site-to-Fault Distance, R_{RUP}	7.4 km / 4.6 miles	12.4 km / 7.7 miles	24.3 km / 15.1 miles

Caltrans structure design practice also requires an increase to spectra due to fault proximity (near-fault factor) and when the site is located over a deep sedimentary basin (basin factor). The near-fault factor applies to this site; the basin factor does not.

We compared the deterministic response spectra for the controlling seismic source identified above to the Caltrans minimum deterministic response spectrum that assumes a maximum moment magnitude 6.9, reverse event occurring at a distance of 4.6 miles.

11.1.2 PROBABILISTIC EVALUATION

The Caltrans ARS Online web-based tool indicates that the 2008 USGS deaggregation of seismic hazard with 5% in 50 years (975-year) fault scenario represents the controlling ground motions from period for 0.00 to 5.00 seconds at this site. A deaggregation was completed using the USGS Unified Hazard Tool[3] to determine the controlling probabilistic fault scenario. The larger of the magnitude M_{mean} vs. M_{mode} values controls for probabilistic seismic design. Table 4 summarizes our deaggregation evaluation.

[3] United States Geological Survey Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/) accessed January 17, 2020.

Table 4: United States Geologic Survey Unified Hazard Tool Deaggregation

Input	
Edition	Dynamic: Conterminous U.S. 2008 (v3.3.1)
Latitude	35.577814
Longitude	-120.835061
Spectral Period	Peak Ground Acceleration
Time History	975 year return period
Site Class	360 m/s (Site Class C)
Output	
Mean Magnitude, M / Site-to-Fault Distance, r	M = 6.5 / r = 19.78 km
Mode Magnitude, M / Site-to-Fault Distance, r	M = 6.1 / r = 10.45 km

11.1.3 RECOMMENDED SEISMIC DATA

The 2008 USGS deaggregation of seismic hazard with 5% in 50 years (975-year) fault scenario represents the controlling ground motions from period for 0.00 to 5.0 seconds at this site. We recommend that structure design be based on the following Caltrans SDC parameters:

- Shear Wave Velocity, VS30: 360 meters per second (1181 fps);
- Soil Profile Type C;
- Magnitude (M): 6.5;
- Peak Ground Acceleration (PGA): 0.40g;
- Controlling Spectra: Probabilistic Spectrum, USGS 5% in 50 years hazard (2008)

We include the recommended design ARS Curve attached as Figure 5.

11.2 SURFACE FAULT RUPTURE

The site does not lie within an Alquist–Priolo Earthquake Fault Zone and no known active faults are mapped within or through the project area. The California Geologic Survey (CGS) considers a fault to be active if it has shown evidence of ground displacement during the Holocene period, defined as the last 11,700 years. According to the CGS, the closest active fault is the Oceanic – West Huasna fault, located about 4.6 miles southwest of the project site. Based on this mapping, the potential for fault rupture at the site is considered to be low.

11.3 LIQUEFACTION POTENTIAL AND SEISMICALLY INDUCED SETTLEMENT

Soil liquefaction is a secondary effect associated with seismic loading. It can occur when saturated, loose to semi-compact, granular soils, or specifically defined cohesive soils, are subjected to ground shaking sufficient to increase pore pressures to trigger liquefaction. In general, liquefaction hazard is most severe within the upper 50 feet of the ground surface. Based on the encountered groundwater level and dense subsurface materials, the potential for liquefaction is considered low.

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. Due to the type of soils/rock encountered, the potential for seismically induced settlement is considered low.

11.4 SEISMIC SLOPE INSTABILITY

We consider the potential for seismically induced slope instability along the channel banks to be low. The potential for seismically induced slides on engineered fill slopes, constructed at typical gradients of 1.5H:1V or flatter, is considered low.

12 FOUNDATION RECOMMENDATIONS

Based on the encountered subsurface materials, the bridge structure support can be achieved using spread footing foundations or cast-in-drilled-hole (CIDH) piles. Based on discussions with MT, spread footings are the preferred support type at the abutments. Spread footing foundation recommendations are provided below.

Spread footing foundations established within "intact" Unit 2 rock are considered adequately stable to support the new bridge structure. No over-riding geologic hazards (e.g., faulting, landslides, subsidence, etc.) were identified by either published geologic mapping or site reconnaissance performed for this study. To achieve long-term security with respect to scour, all permanent bridge spread footing foundations are recommended to be established with at least 3 ft of penetration into the underlying intact rock.

Geotechnical conditions that will require particular consideration in new spread footing foundation design and construction include:

- variations in the rock surface within footing excavations at the abutments,
- potential disturbance of bearing materials due to removal of existing foundations, and
- the presence of ground/surface water.

Site conditions also considered suitable for cast-in-drilled-hole (CIDH) piles. Such piles would achieve support within the underlying "weathered" rock through side friction and designed with assured penetration of bearing materials for consideration of long term security with respect to scour.

Steel pipe piles (driven open-end) and steel H-piles could also be considered at this site. However, erratic driving conditions may occur depending on the variability of weathering in the underlying rock unit. If very limited penetration into the rock unit occurs (i.e., point bearing only), it will result in little lateral or tensile resistance. We do not recommend driven concrete piles at this site. Such piles would likely experience very hard driving within portions of the rock unit (resulting in possible pile damage). Such piles would likely require undersize drilling to assist driving in order to achieve adequate pile penetration.

12.1 SHALLOW FOUNDATIONS

12.1.1 FOUNDATION DATA AND LOADING

Foundation data and loading for spread footing foundations was provided by MT and is presented in Tables 5 through 7 below.

Table 5: Foundation Data

Support Location	Design Method	Finished Grade Elevation (ft)	Bottom of Footing Elevation (ft)	Footing Size (ft)		Permissible Settlement Under Service Load* (inches)
				B	L	
Abut 1	LRFD	1075.0	1065.0	11.5	28.0	2.0
Abut 2	LRFD	1079.5	1070.0	11.5	28.0	2.0

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

Table 6: LRFD Service - I Limit State Load for Controlling Combination[1]

Support Location	Total Load					Permanent Load[2]				
	P Total (Kips) Net	M_X (kip-ft)	M_y (kip-ft)	V_X (kips)	V_Y (kips)	P Total (Kips) Net	M_X (kip-ft)	M_y (kip-ft)	V_X (kips)	V_Y (kips)
Abut 1	530	-1540	N/A	N/A	305	355	-180	N/A	N/A	220
Abut 2	620	1230	N/A	N/A	70	420	30	N/A	N/A	150

1) Controlling load combination is the one resulting in the highest ratio of qg,max/qR for foundations on rock.
2) See table 3.4.1-2 in the AASHTO LRFD Bridge Design Specifications for components of permanent load. Total and Permanent Loads are NET for Service-I Limit State.

Table 7: LRFD Strength, Construction and Extreme Event Loads for Controlling Load Combinations

Support Location	Strength/Construction Limit State (Controlling Group)[1]					Extreme Event Limit State (Controlling Group)[1]				
	P Total (Kips) Net	M_X (kip-ft)	M_y (kip-ft)	V_X (kips)	V_Y (kips)	P Total (Kips) Net	M_X (kip-ft)	M_y (kip-ft)	V_X (kips)	V_Y (kips)
Abut 1	1775	2680	N/A	N/A	70	N/A	N/A	N/A	N/A	N/A
Abut 2	1610	2130	N/A	N/A	15	N/A	N/A	N/A	N/A	N/A

3) Controlling load combination is the one resulting in the highest ratio of qg,max/qR for foundations on rock.

12.1.2 FOUNDATION DESIGN RECOMMENDATIONS AND PILE DATA TABLE

The foundation recommendations at the abutments are summarized in Table 8, and the recommended permissible bearing stress/pressure is presented Table 9.

Table 8: Foundation Design Recommendations for Spread Footing

Support Location	Footing Size (ft)		Bottom of Footing Elevation (ft)	Minimum Footing Embedment Depth[4] (ft)	Total Permissible Support Settlement (inches)	Service Limit State	Strength Construction Limit State Φ_b=0.45	Extreme Event Limit State Φ_b=1.00
	L	B				Permissible Net Constant Stress[2] (ksf)	Factored Gross Nominal Bearing Resistance[3] (ksf)	Factored Gross Nominal Bearing Resistance[3] (ksf)
Abut 1	11.5	28.0	1065.0	3.0	2.0	24.0	10.0	N/A
Abut 2	11.5	28.0	1070.0	3.0	2.0	24.0	10.0	N/A

1) Controlling load combination is the one resulting in the highest ratio of $q_{g,max}/q_R$ for foundations on rock.
2) For Service-I Limit State, controlling load combination is the one resulting in the highest ratio of $q_{g,max}/q_R$ for foundations on rock. Permissible Net Contact Stresses were calculated for controlling load combinations.
3) For Strength, Construction, and Extreme Event Limit States, controlling is the one resulting in the highest ratio for $q_{g,max}/q_R$ foundations of rock, Factored Gross Nominal Bearing Resistance were calculated for controlling load combinations.
4) The embedment depth.

Table 9: Spread Footing Data Table[1]

Support Location	Service[2] Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction[3] Factored Gross Nominal Bearing Resistance O_b = 0.45 (ksf)	Extreme Event[3] Factored Gross Nominal Bearing Resistance O_b = 1.00 (ksf)
Abut 1	24.0	10.0	N/A
Abut 2	24.0	10.0	N/A

1) Controlling load combination is the one resulting in the highest ratio of $q_{g,max}/q_R$ for foundations on rock.
2) For Service-I Limit State, controlling load combination is the one resulting in the highest ratio of $q_{g,max}/q_R$ for foundations on rock. Permissible Net Contact Stresses were calculated for controlling load combinations.
3) Controlling load combination for Strength, Construction, and Extreme Event is the one resulting in the highest ratio for $q_{g,max}/q_R$ foundations of rock.

12.1.3 COMPRESSIVE RESISTANCE

Table C10.6.2.6.1-1 of the AASHTO Bridge Design Specifications indicates presumptive bearing resistance at the Service Limit State in the ordinary range of 16 to 24 ksf for intact rock with medium hard in place consistency. The indicated bearing resistance values are limited to 1-inch (or less) of settlement and apply only at the service limit state. For the purpose of bridge foundation design, the intact sedimentary rock encountered in the borings completed for this study is consistent with this description. We conservatively recommend a factored gross nominal bearing resistance of 10.0 ksf for spread footing foundations with the bottom of footing established at least 3 ft into "intact" rock. The planned bottom of footing elevations are expected to meet the rock penetration criteria.

Eccentricity should be checked by the structural designer.

12.1.4 SETTLEMENT

Foundations bearing on "intact" rock are expected to experience nominal settlement (less than 1 inch) and settlement is expected to be substantially completed during construction. We expect differential settlement to be less than one-half of the total settlement. Total settlement is also dependent on the contractor's means and methods.

12.1.5 SLIDING RESISTANCE

To evaluate spread footing foundation sliding resistance for Strength and Extreme Limit State load combinations consistent with Section 10.6.3.4 of the 2012 AASHTO LRFD Bridge Design Specifications and current Caltrans amendments, use the following:

- A friction angle (ϕ_f) equal to 38˚ corresponding to a friction factor (tan δ = tan ϕ_f) of 0.78 for cast in-place concrete foundations bearing on native, "intact" rock or plain structural concrete. Use a resistance factor (ϕ_τ) of 0.80 for shear resistance between the bottom of the cast-in-place concrete footing and bearing material.
- No passive resistance is allowed.

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

12.1.6 CONSTRUCTION CONSIDERATIONS SHALLOW FOUNDATIONS

The contractor will need to take into account the following considerations during the footing excavation:

- Variations in the rock surface within footing excavations at the abutments,
- Difficult excavation due to variations in the weathering of the underlying sedimentary rock,
- Uniaxial Compressive Strengths of tested sedimentary rock (shale) cores vary from 400 psi to 1,200 psi,
- Potential disturbance of bearing materials due to removal of existing foundations, and
- Difficult excavation due to groundwater/seepage during the wet season within the footing excavation.

Interference of the existing bridge foundations with new bridge foundations and disruption from removing the existing facility may require increased footing penetration, or other consideration.

At all support locations, construct spread footing foundations on intact/undisturbed bearing material at the bottom of the excavation within clean and dry excavations. Place structural footing concrete neat (without forming), against trimmed walls and intact rock at the bottom of the footing excavation.

All footing foundation excavations are to be inspected and approved by the geotechnical engineer or engineering geologist. The inspections are to be made after the excavation has been completed to the bottom of footing elevations and prior to placing concrete or rebar in the excavations.

Any exposed open joint/fractures should be evaluated by the geotechnical engineer or engineering geologist with respect to bearing/stability considerations and cleaned/surfaced-grouted, if necessary.

Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed and replaced to the bottom of the footing elevation with plain structural concrete.

If it is necessary to deepen footing excavations to engage suitable bearing materials, it is acceptable to backfill with plain structural concrete to plan footing grade, up to a depth of 3 ft below the footing, with the approval of a geotechnical engineer or engineering geologist.

12.2 APPROACH FILLS

12.2.1 EARTHWORK

Site grading and earthwork should be performed in accordance with Section 17 and Section 19 of Caltrans "Standard Specifications", or San Luis Obispo County Standards.

12.2.2 FILL MATERIAL

Construct embankment and place new fill in accordance with Caltrans "Standard Specifications", including at least 95% relative compaction (CTM 216) on all fill within 150 feet of bridge abutments. Where new fill is placed against an existing slope or when constructing half the embankment width at a time, prepare the slope by cutting into it at least 6 feet horizontally and below any loose/soft or otherwise unsuitable materials as the new embankment is placed in layers (consistent with Section 19 of Caltrans "Standard Specifications").

Any imported fill should be approved by the soils engineer, should have 100% passing 3-in sieve and have low expansion potential [Expansion Index (EI) < 50 and Sand Equivalent (SE) > 20]. Imported fill used at and below subgrade level should also be required to meet or exceed that of the design R-value (See Section 13.2). In general, all fill material should be free of debris and organic material.

Expansive soil (EI ≥ 50 and SE ≤ 20) should not be used as fill in any portion of the abutment wall/wingwall and the abutment front slope.

12.2.3 SLOPE GEOMETRY AND STABILITY

We consider the potential for seismically induced slope instability along the channel banks to be low, limited to some minor distortion. The potential for seismically induced slides on engineered fill slopes, constructed at typical gradients of 1.5H:1V or flatter, is considered low.

12.2.4 SETTLEMENT

Due to the presence of rock at a shallow depth, we do not anticipate significant immediate or long-term embankment settlement.

13 ADDITIONAL CONSIDERATIONS

13.1 LATERAL EARTH PRESSURES

The approach fill material behind the abutments and wingwalls is expected/recommended to meet Structure Backfill requirements consistent with Caltrans Standard Specifications. Use of the equivalent fluid weights (EFWs) shown in Table 10 are recommended to design the abutment and wing walls (assuming fully drained and level backfill conditions).

Table 10: Recommended Equivalent Fluid Weight (EFW)

Condition	Static		Incremental Seismic	
	Coefficient k (dim.)	EFW (pcf)	Coefficient Δk (dim.)	ΔEFW_{EQ} (pcf)
Active	0.28	34	0.06	8
At-Rest	0.44	53	NA	13

The EFW values shown above are consistent with Caltrans standards/practice and assume:

- Level backfill condition;
- Caltrans Structure Backfill with soil unit weight (γ) = 120 pcf and minimum angle of internal friction (ϕ) = 34°;
- Horizontal seismic acceleration coefficient (k_h) ≤ 0.2;
- Vertical seismic acceleration coefficient (k_v) = 0.0; and
- Drainage behind walls is placed in accordance with Caltrans Standard Plans and Specifications.

13.1.1 STATIC LATERAL EARTH PRESSURE

Use the static active EFW_A equal to 34 pcf for walls that can deflect or move sufficiently to reach minimum active condition (Caltrans allows use of active earth pressure for embankment behind seat-type abutments). With Structure Backfill (i.e., equivalent to a medium dense sand), use of the active equivalent earth pressure assumes the wall is free to rotate at least 0.002 times the wall height to mobilize the active condition. Use the static at-rest EFW_O equal to 53 pcf for walls that are restrained at the top and do not deflect or move (e.g., behind diaphragm-type abutments).

The resultant earth pressure (P_{static}) for each foot of retained soil can be estimated as $0.5(EFW_A)H^2$ for active condition and $0.5(EFW_O)H^2$ for at-rest condition (whichever controls), where H is the height of the wall in feet measured from the base of the wall. Use a triangular pressure distribution and apply the controlling static resultant earth pressure at a distance of H/3 from the base of the wall.

The static active and at-rest earth pressure coefficients were calculated using the Rankine and Coulomb equations presented in Section 5 of Caltrans Bridge Design Specifications (BDS, August 2004). The Rankine and Coulomb equations yield the same value for the active coefficient (k_a) when the friction angle between the backfill material and back of wall (δ) is equal to zero.

13.1.2 SEISMIC LATERAL EARTH PRESSURE

Use the incremental seismic equivalent fluid weight (ΔEFW_{EQ}) equal to 8 pcf for active condition and 13 pcf for at-rest condition. The active seismic coefficient (K_{AEQ}) was calculated using the Mononabe-Okabe (M-O) equation presented in Appendix A11 of AASHTO LRFD Bridge Design Specifications (6th Edition). The incremental active seismic coefficient (Δk_{AEQ}) was estimated by subtracting the active static coefficient from the active seismic coefficient (i.e., $\Delta k_{AEQ} = K_{AEQ}$ -

k_A). The incremental at-rest seismic coefficient (Δk_{OEQ}) was estimated using an increase ratio similar to the active condition.

The ΔEFW_{EQ} is equal to the controlling seismic coefficient (Δk_{EQ}) multiplied by the soil unit weight (γ). In the M-O equation, we used a horizontal seismic acceleration coefficient (k_h) value of 0.13 (i.e., approximately one-third of the site PGA consistent with Caltrans Geotechnical Manual, April 2016).

The resultant incremental seismic lateral soil pressure (ΔP_{EQ}) for each foot of retained soil can be estimated as $0.5(\Delta EFW_{AEQ})H^2$ for active condition and $0.5(\Delta EFW_{OEQ})H^2$ for at-rest condition (whichever controls), where H is the height of the wall in feet. Use a uniform pressure distribution and apply the magnitude of the resultant at 0.5H from the base of the wall. The total seismic load is equal to the resultant of the incremental seismic earth pressure added to the resultant of the static earth pressure (i.e., $P_{EQ} = P_{static} + \Delta P_{EQ}$).

As noted in the Caltrans Seismic Design Criteria (SDC), the maximum passive pressure is 5.0 ksf, which must be used with the proportionality factor presented in Section 7.8.1 of the SDC. Assuming that backfill at the abutments meets Caltrans criteria for structure backfill, SDC Section 7.8 criteria for initial abutment soil stiffness (50 kips/inch/ft) should be applicable.

13.1.3 SURCHARGE LOADS

For surcharge loads, apply an additional uniform lateral load behind the wall that is the greater of the values shown in Table11 for the applicable earth pressure condition.

Table 11: Surcharge Loads

Active Condition	At-Rest Condition
0.3-times the design surcharge pressure	0.44-times the design surcharge pressure
0.3-times a minimum surcharge pressure of 240 psf	0.44-times a minimum surcharge pressure of 240 psf

The minimum surcharge pressure of 240 psf represents an equivalent height of soil equal to 2 ft where the unit weight of soil is 120 pcf. For equivalent heights of soil for highway loadings on abutments and retaining walls refer to Section 3.11.6.4 Live Load Surcharge of the 2012 AASHTO LRFD Bridge Design Specifications (6th Edition).

13.2 PAVEMENT STRUCTURAL SECTION AND ROADWAY SUBGRADE

R-value tests (CTM 301) will be completed on bulk samples of anticipated subgrade soils; the test results will be sent upon completion. For now, a design R-value of 25 is being assumed for proposed pavement structural section design.

New flexible pavement structural section alternatives calculated in accordance with Caltrans flexible pavement design methods for various Traffic Index (TI) values at a design R-value of 25 are shown in Table 12. If needed, recommendations for a project specific new pavement structural section can also be provided based on a defined project TI value.

Table 12: New Pavement Structural Sections (R-value = 25)

Section	Material	Traffic Index (TI)		
		5.0	6.0	7.0
Hot Mix Asphalt (HMA) over Class 2 Aggregate Base (AB)	HMA (feet)	0.20	0.25	0.30
	AB (feet)	0.65	0.80	0.95

The asphalt pavement thicknesses shown above are minimum depths and incorporate a 0.2-ft Gravel Equivalent factor of safety in accordance with Caltrans flexible pavement design methods. Other non-Caltrans flexible pavement structural sections – typically involving variations in AB thicknesses – which satisfy basement soil requirements are available and can be provided, if desired.

Design by the Caltrans method presumes materials and construction in accordance with Caltrans “Standard Specifications”, including 95% relative compaction (CTM 216) on all materials within 30 inches of finished grade. Inability to achieve the required compaction on the scarified materials may be used as a field criterion to identify areas requiring additional removal and/or re compaction.

The subgrade soils should be field reviewed with respect to uniformity and suitability by the soils engineer. Any unsuitable material, including clay and loose or disturbed soils, should be removed to full depth and replaced with granular native soil or Class 2 Aggregate Base compacted to at least 90% relative compaction (CTM 216).

The above pavement design assumes that free water will be absent from the structural section. Suitable surface drainage is of particular importance to limit subgrade saturation and excess free water.

13.3 CONSTRUCTION CONSIDERATIONS

This section is provided to help identify relevant Standard Specifications and subsurface conditions that may be encountered in the field during construction. For the project described herein, it is recommended that the foundation report, LOTB, and any subsequent addenda be included with project documents during the bidding process for reference purposes.

13.3.1 EXCAVATION AND SHORING

We expect that excavation of soil and rock to the indicated foundation depths can be achieved using typical heavy-duty construction equipment and that excavation of weathered rock within footing limits and depths indicated above will be locally difficult, but achievable without blasting. Use of air tools may be necessary to excavate rock. Rock blasting may disrupt/degrade the integrity of the surrounding rock and therefore, should not be permitted.

Existing soils overlying the rock unit are consistent with Cal OSHA Type C soil classification. The contractor is responsible for design and construction of excavation sloping and shoring in accordance with Cal/OSHA requirements, including verifying soil type in open excavations, and to protect existing structures, utilities and other facilities during construction.

13.3.2 DEWATERING

Adequate construction de-watering for the abutment excavations is expected to be achievable during dry season construction (approximately June through October) by means of diking/diversion of surface water (if present) and the use of sump pumps, but could require heavy pumping. Temporary diversion/piping of all surface water around/through the site is considered prudent.

Winter or spring construction can expect higher water surface level in the channel and may also encounter higher/perched groundwater levels, possibly under head, and require additional controls. The contractor is responsible for dewatering and/or diking diversion design and construction methods.

13.3.3 SEAL COURSE

Footing construction requires concrete to be poured neat without forming against undisturbed rock. If excavations are unable to be dewatered, the use of a tremie seal course can be considered for spread footing foundations.

Caltrans indicates a minimum 2-foot thick seal course. The thickness of the seal course should be determined by the design engineer based on consideration of the foundation type, anticipated maximum hydrostatic head, and the permissible highest elevation of the top of the reinforced concrete footing. A chart for determining the seal course thickness as presented in the Caltrans Foundation Manual (2008, Revision 01 December 2010) is included in Appendix B.

A seal course, if used below the bottom of the structural footing, will need to extend beyond the footing footprint based on a 0.5:1 (horizontal:vertical distance) plane projected from the bottom of the outside edge of the structural footing.

14 RISK MANAGEMENT

Within our profession it is recognized that the risks of design, construction, and maintenance-related problems associated with civil engineering works are typically higher and result in increased overall project cost when the geotechnical engineer of record is not retained to provide supplemental services. For this project, CAInc should provide the following supplemental geotechnical services:

- Review and provide written comments on the final plans and specifications, insofar as they rely upon this report, prior to construction bidding to verify consistency with the recommendations contained herein; and,
- Review footing excavations during construction in order to confirm anticipated bearing materials and provide additional or alternate recommendations if necessary.

Should there be significant change in the project or should soil/rock conditions different from those described in this report be encountered during construction, this office should be contacted/notified for evaluation and supplemental recommendations as necessary or appropriate.

CAInc cannot be responsible for any other parties' interpretation of our report and recommendations contained herein, as well as subsequent addendums, letters, and discussions. If others perform the construction observation, they should review this report and either accept the conclusions and recommendations herein as their own or provide alternative recommendations.

15 LIMITATIONS

The conclusions and recommendations of this study are professional opinion based upon the indicated project criteria and the limited data described herein. It is recognized there is potential for variation in subsurface conditions and modification of conclusions and recommendations might emerge from further, more detailed study.

This report is intended only for the purpose, site location, and project description indicated and construction in accordance with Caltrans practice.

As changes in appropriate standards, site conditions and technical knowledge cannot be adequately predicted; review of recommendations by this office for use after a period of two years is a condition of this report.

A review by this office of any foundation and/or grading plans and specifications or other work product insofar as they rely upon or implement the content of this report, together with the opportunity to make supplemental recommendations as indicated therefrom is considered an integral part of this study and a condition of recommendations.

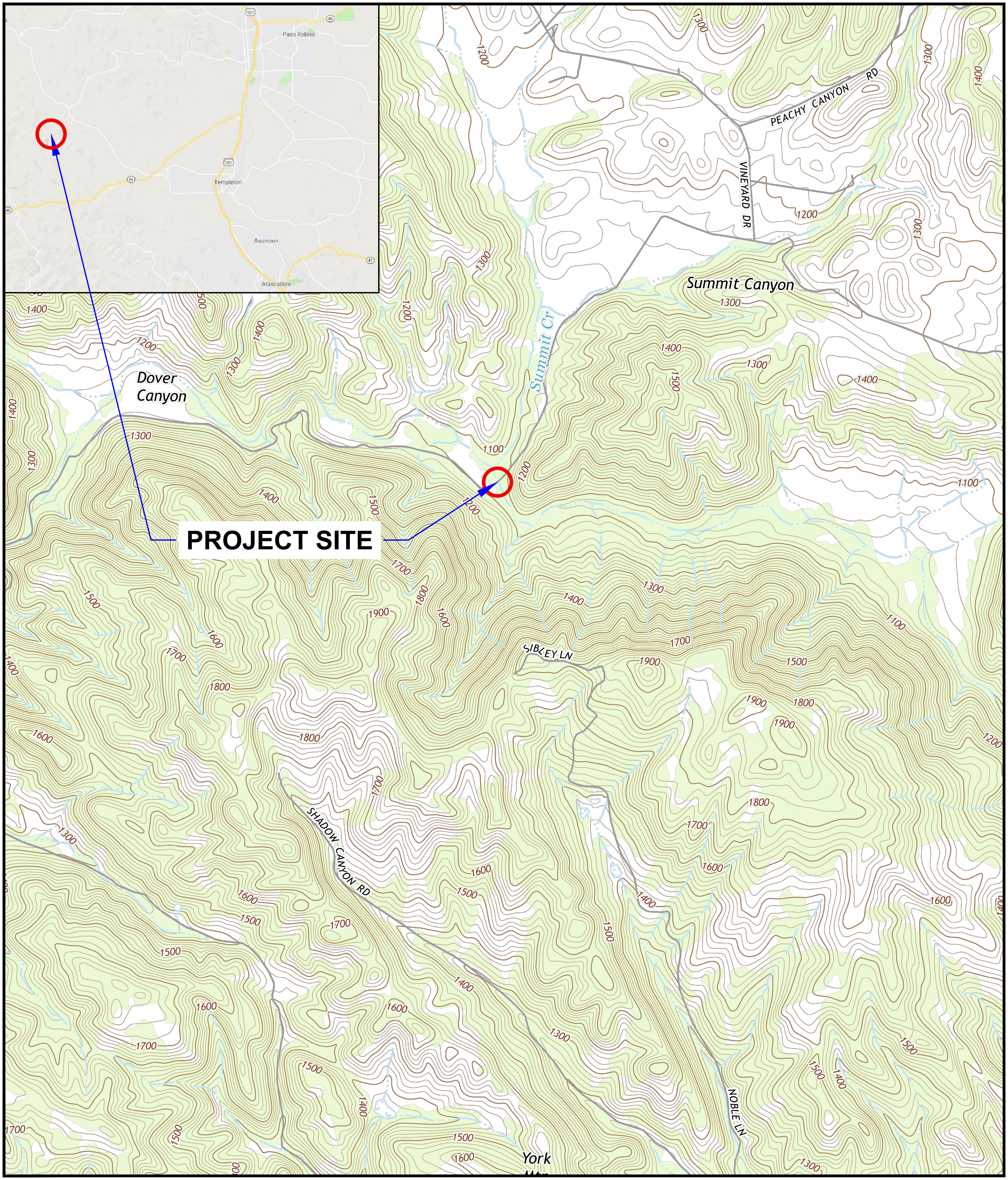
Subsequently defined construction observation procedures and/or agencies are an element of work, which may affect supplementary recommendations.

Should there be significant change in the project or should soil/rock conditions different from those described in this report be encountered during construction, this office should be notified for evaluation and supplemental recommendations as necessary or appropriate.

Opinions and recommendations apply to current site conditions and those reasonably foreseeable for the described development--which includes appropriate operation and maintenance thereof. They cannot apply to site changes occurring, made, or induced, of which this office is not aware and has not had opportunity to evaluate.

The scope of this study specifically excluded sampling and/or testing for, or evaluation of the occurrence and distribution of, hazardous substances. No opinion is intended regarding the presence or distribution of any hazardous substances at this or nearby sites.

FIGURES



NORTH

Map Source:
USGS 7.5' Topographic Maps 2015, YORK MOUNTAIN, California, Scale 1:24000.

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(916) 455-4225

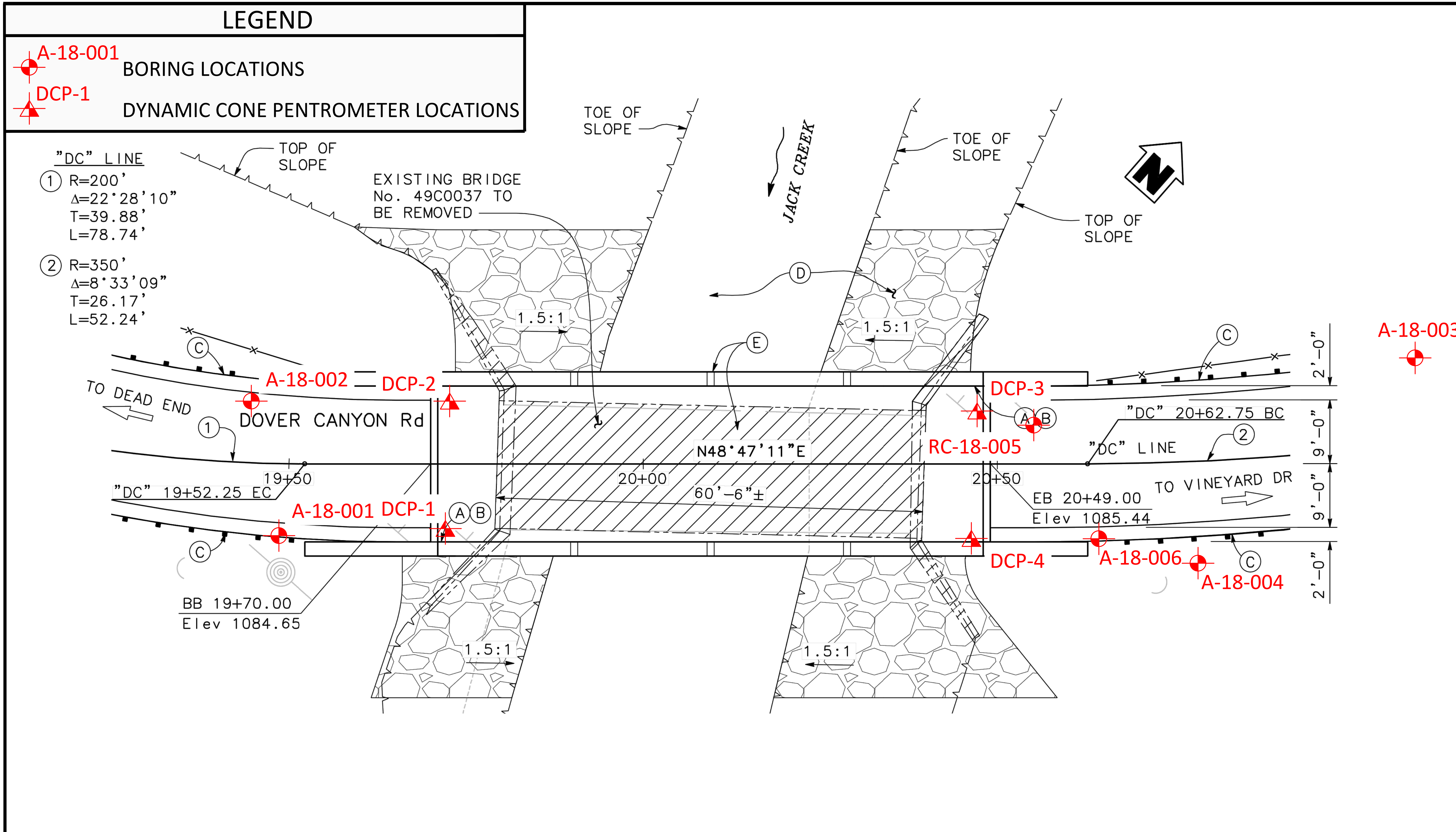
Dover Canyon at Jack Creek Bridge Replacement

San Luis Obispo County, CA

Figure 1
Vicinity Map

Proj. No:	17-371.1
Scale:	1"= 2,000'
Date:	3/29/18

Path: /Users/S Crawford/Box Sync/Projects/17-375.1 Dover Canyon Road at Jack Creek Bridge Replacement Project/CAD/Figures/DWG/17-371.1 Dover Canyon Road Bridge Figures.dwg Plot Date: Mar 29, 2018 at 4:08pm



Source:
"Dover Canyon at Jack Creek Bridge Replacement Project," General Plan. Mark Thomas, Inc. Received January 29, 2020.

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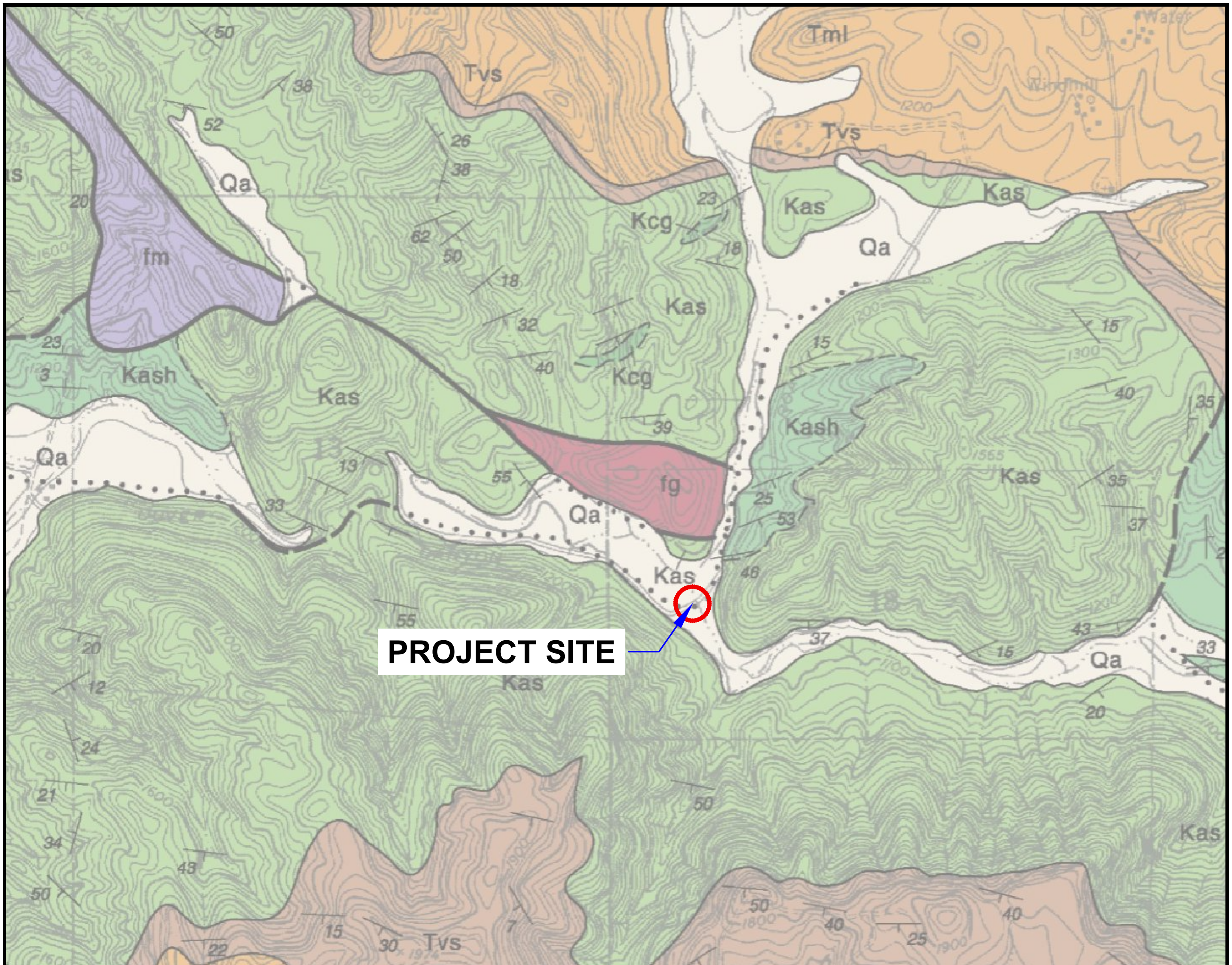
Dover Canyon Road at Jack Creek Bridge

San Luis Obispo County

Figure 2
Exploration Location Map

Proj. No: 17-375.1
Scale: 1"=12.5'
Date: 1/30/2020



GEOLOGIC FORMATIONS LEGEND

Qa

SURFICIAL SEDIMENTS

Qa *Alluvial sand & gravel*

fm | fg

FRANCISCAN ASSEMBLAGE

Age, Jurassic-Cretaceous

fm *Melange mixture composed of intensely sheared, dark gray claystone that includes numerous tectonic fragments of graywacke, greenstone (meta-basalt), varicolored chert, (jasperized) and blueschist*

fg *Greenstone*

Kas | Kash | Kcg

MARINE CLASTIC SEDIMENTARY ROCKS

(Atascadero Formation of Fairbanks, 1904, Sieders, 1989, lithified sedimentary rocks; age, late Cretaceous (Campanian?)

Kas *Sandstone, light brown, thick-bedded, locally pebbly, much shattered*

Kash *Shale, dark gray, micaceous clay shale, thin-bedded with fine-grained olive-gray sandstone*

Kcg *Cobble conglomerate, gray, with cobbles of meta-volcanic porphyries, granitic rocks and quartzite*

CONTACT

(Dashed where approximately located)

FAULT

(Dashed where approximately located; dotted where concealed)

NORTH

Map Source:

Dibblee, T.W. and Minch, J.A.: Dibblee Geological Foundation Map - Geologic map of the York Mountain quadrangle, San Luis Obispo, CA; 2006; scale 1:24,000.

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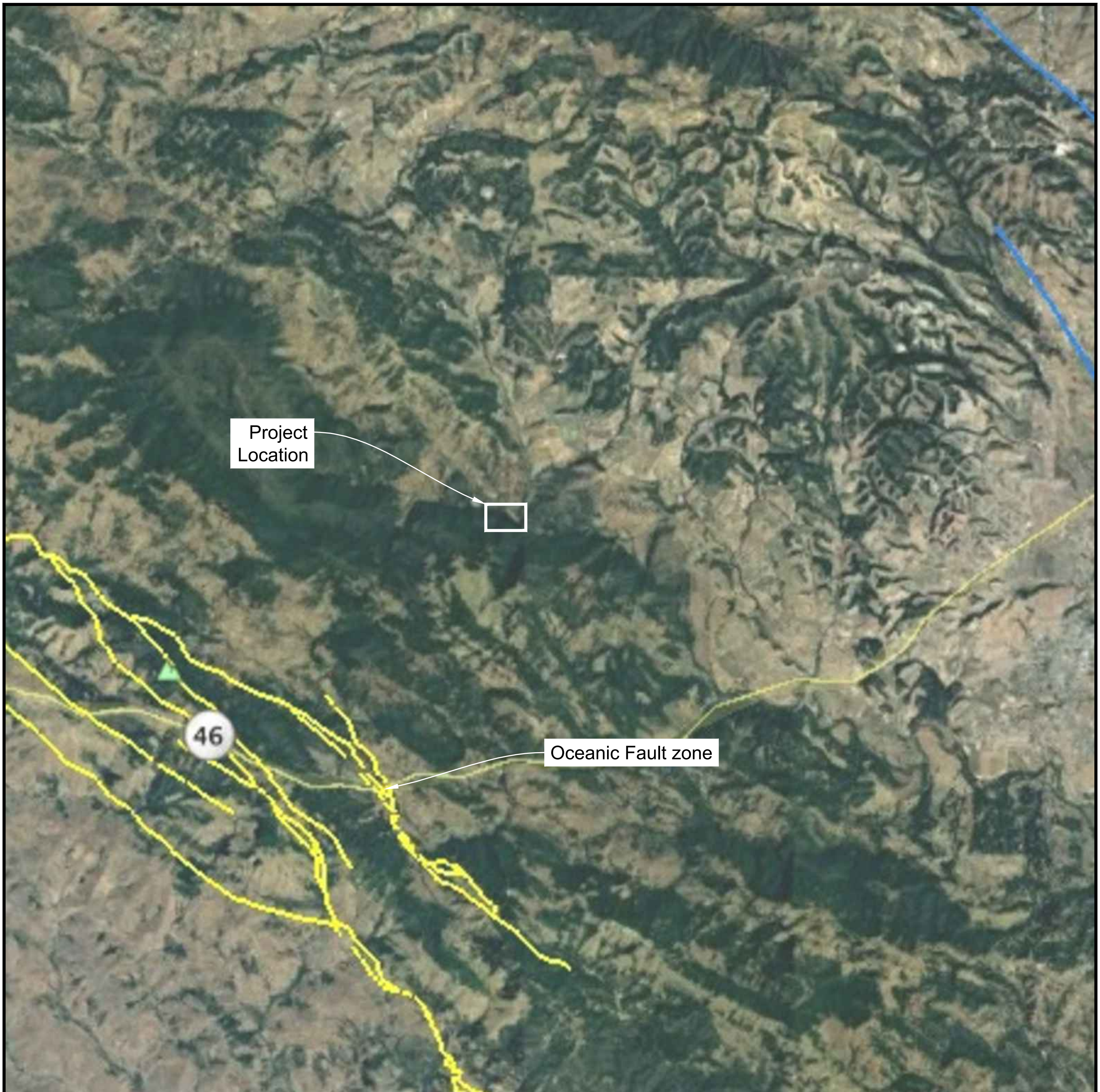
Dover Canyon at Jack Creek Bridge Replacement

San Luis Obispo County, CA

Figure 3
Geologic Map

Proj. No:	17-371.1
Scale:	1"= 3,000'
Date:	6/26/18

Path: /Users/S Crawford/Box Sync/Projects/17-375.1 Dover Canyon Road at Jack Creek Bridge Replacement Project/CAD/Figures/DWG/17-371.1 Dover Canyon Road Bridge Figures.dwg Plot Date: Mar 29, 2018 at 4:08pm



LEGEND

Quaternary Fault (Age)	Quaternary Fault (Age)	Location
<150 years	<750,000 years	Well Constrained
<15,000 years	<1.6 million years	Moderately Constrained
<130,000 years		Inferred

North

Source: Google Earth Pro with USGS Fault Map Overlay

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Dover Canyon at Jack Creek Bridge Replacement

San Luis Obispo County, CA

Figure 4
Fault Activity Map

Proj. No: 17-375.1
Scale: 1"=9,000'
Date: 6/26/18

SEISMIC DESIGN DATA

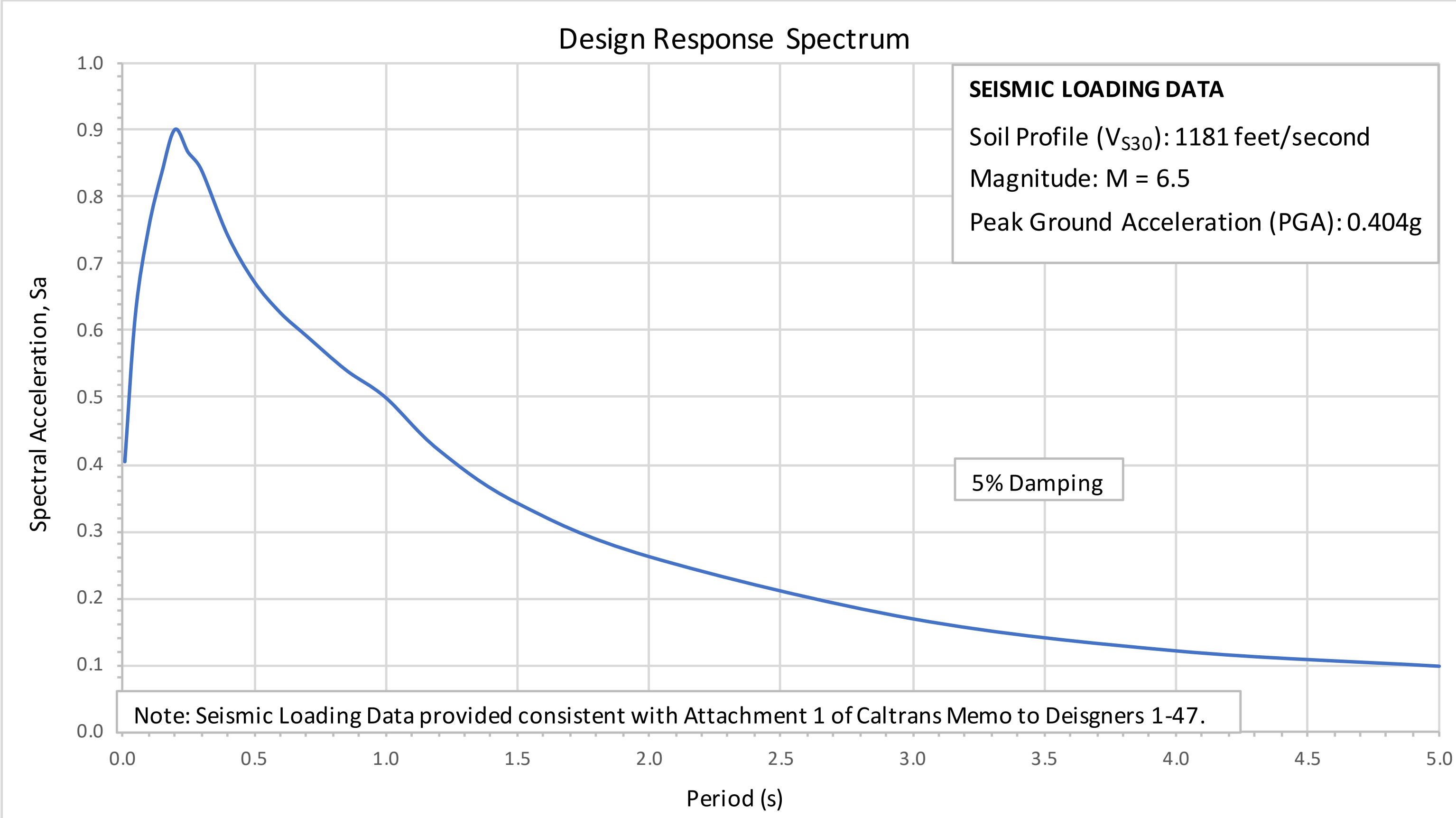
Dover Canyon Road at Jack Creek Bridge (Bridge Number 49C0037)

San Luis Obispo County, California

Caltrans ARS Online Version: V2.3.09

Date Accessed: 1/17/2020

Period (s)	Spectral Acceleration, Sa (g)
0.010	0.404
0.050	0.624
0.100	0.753
0.150	0.836
0.200	0.901
0.250	0.867
0.300	0.841
0.400	0.743
0.500	0.674
0.600	0.627
0.700	0.592
0.850	0.541
1.000	0.500
1.200	0.422
1.500	0.342
2.000	0.262
3.000	0.169
4.000	0.121
5.000	0.098



The Design Response Spectrum is the upper envelope of the deterministic and probabilistic repsonse spectrum, but not less than the Minimum Deterministic Spectrum for California. The deterministic spectrum is obtained by using the average of the 2008 Campbell-Bozorgnia and the 2008 Chiou-Youngs ground motion prediction equations. Probabilistic response spectrum is obtained for the 5 percent probability of exceedance in 50 years from the 2008 USGS Interactive Deaggregation web tool.

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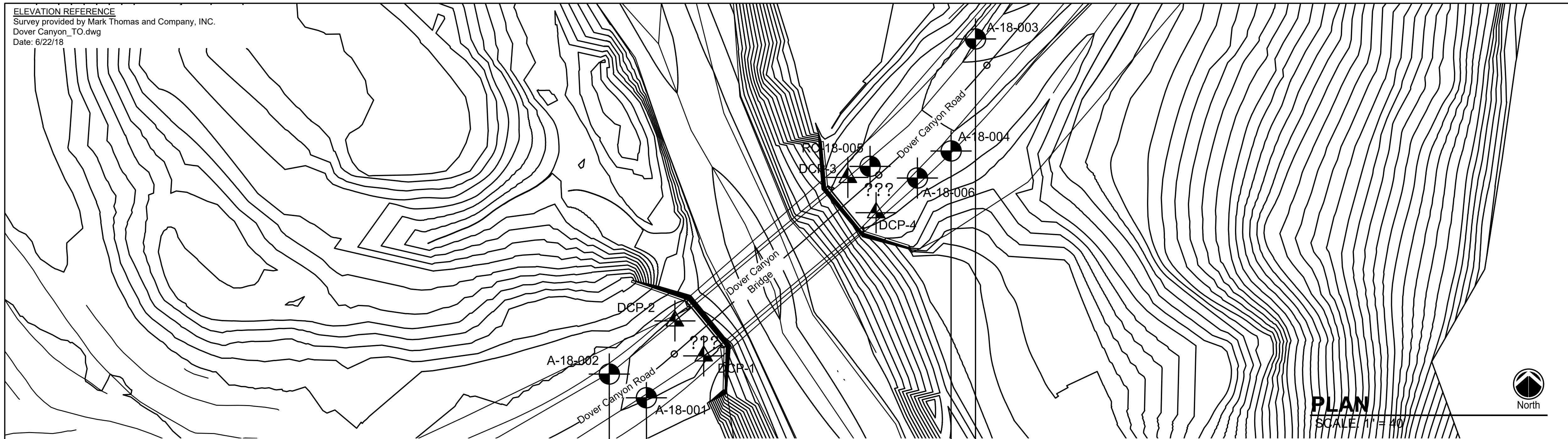
Site Latitude: 35.577814
Site Longitude: -120.835061

FIGURE 5

APPENDIX I

Log of Test Borings

ELEVATION REFERENCE
Survey provided by Mark Thomas and Company, INC.
Dover Canyon_TO.dwg
Date: 6/22/18



Dist	COUNTY	ROAD NO.	JOB NO.	SHEET No.	TOTAL SHEETS
05	SLO			01	03

REGISTERED CIVIL ENGINEER — DATE 1/30/2020

PLANS APPROVAL DATE 1/30/2020

REGISTERED PROFESSIONAL ENGINEER BENJAMIN D. CRAWFORD No. 68457 CIVIL STATE OF CALIFORNIA

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services
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1100 Corporate Way
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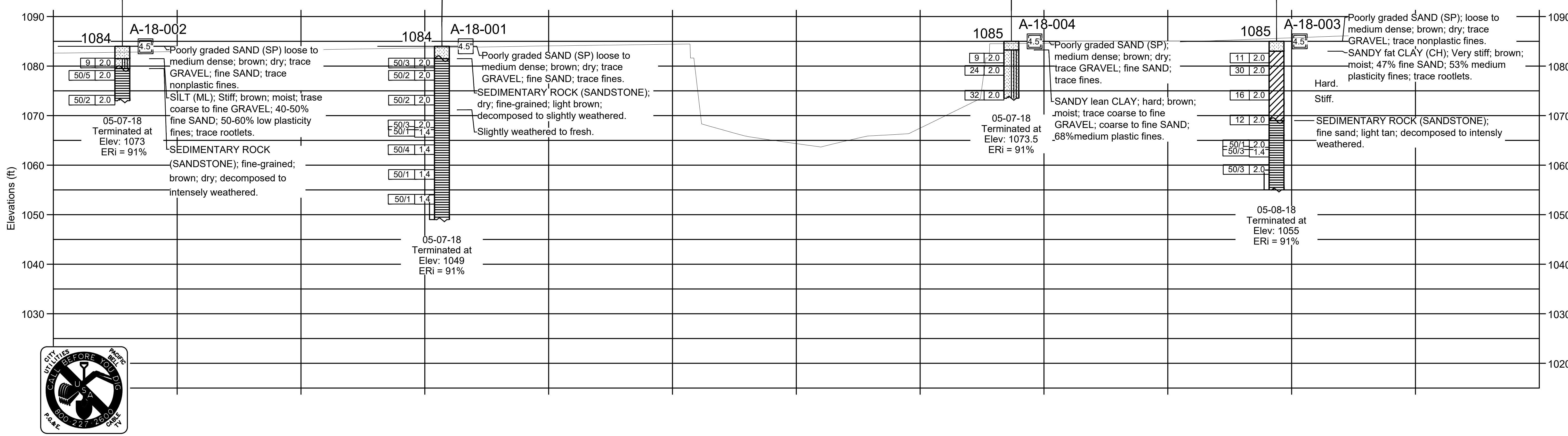
Sacramento, CA 95831
(916) 455-4225

NOTES:
1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010). See Log of Test Borings No. 3, "Soil 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. "2.0 inch sampler": ID=2.0 inch, OD=2.5 inch. Both driven in same manner as SPT ("1.4 inch") sampler.
4. If laboratory tests are not shown as being performed, the soil descriptions presented on the LOTB are based solely on the visual practices described in this Manual.
5. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 2010). Where less than 0.5 feet of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
6. Consistency of soils shown in ( ) where estimated.
7. Groundwater surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
8. Groundwater elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
9. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".
10. Boring locations on the cross section view are not at the correct startions, they are at the correct elevations.



CALL BEFORE YOU DIG
1-800-227-2600

PROFILE HORIZ: 1"=20' VERT: 1"=20'

DESIGN	BY Barrett Updegraff	CHECKED Benjamin Crawford		BRIDGE NO. 49C0037
FIELD INVESTIGATION	BY Hailey Wagenman			POST MILES

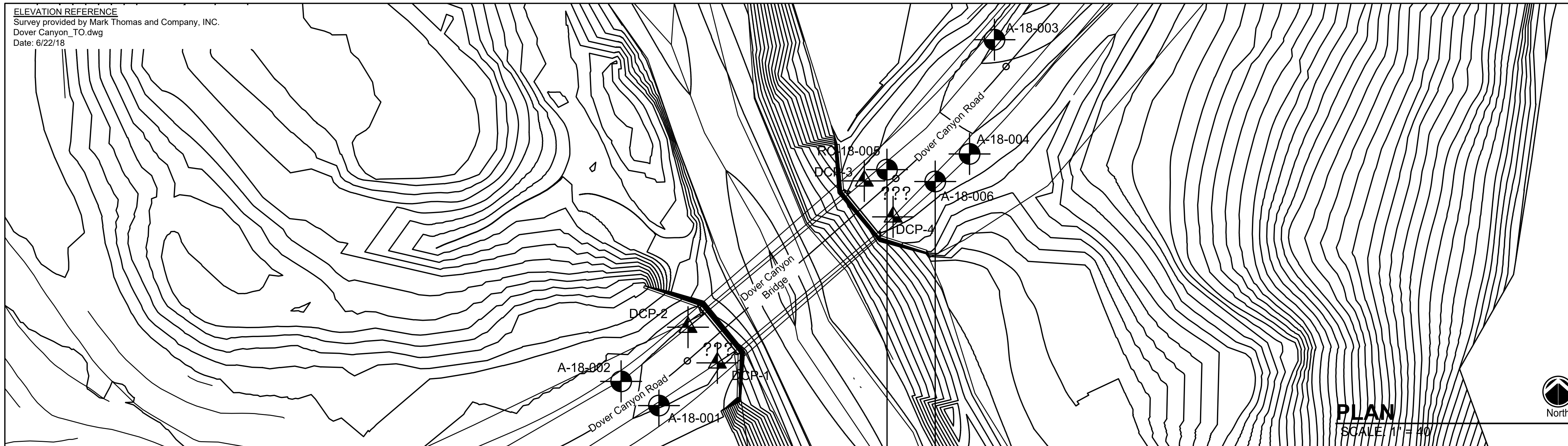
PREPARED FOR THE
SAN LUIS OBISPO COUNTY
PUBLIC WORKS DEPARTMENT
976 OSOS STREET, ROOM 206, SAN LUIS OBISPO, CALIFORNIA 93408

DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT

LOG OF TEST BORINGS 1 OF 3

\\MAC\HOME\BOX\PROJECTS\17-375.1 DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT\CAD\LOTB 11X17\DWG\LOTB.DWG, 8/28/2018 2:58 PM, ----, BARRETTUPDEGRAFF

ELEVATION REFERENCE
Survey provided by Mark Thomas and Company, INC.
Dover Canyon_TO.dwg
Date: 6/22/18



Dist	COUNTY	ROAD NO.	JOB NO.	SHEET No.	TOTAL SHEETS
05	SLO			02	03

REGISTERED CIVIL ENGINEER — DATE 1/30/2020

PLANS APPROVAL DATE 1/30/2020

REGISTERED PROFESSIONAL ENGINEER BENJAMIN D. CRAWFORD No. 68457 CIVIL STATE OF CALIFORNIA

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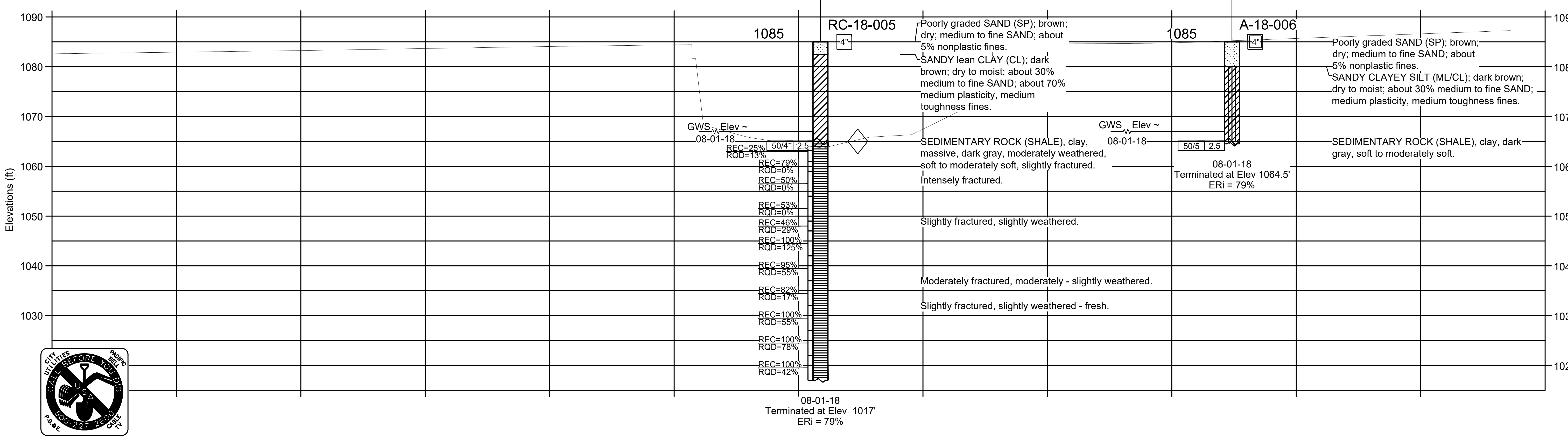
NOTES:

1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010). See Log of Test Borings No. 3, "Soil 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. "2.0 inch sampler": ID=2.0 inch, OD=2.5 inch. Both driven in same manner as SPT ("1.4 inch") sampler.
4. If laboratory tests are not shown as being performed, the soil descriptions presented on the LOTB are based solely on the visual practices described in this Manual.
5. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 2010). Where less than 0.5 feet of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
6. Consistency of soils shown in ( ) where estimated.
7. Groundwater surface (GWS) elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
8. Groundwater elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
9. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".
10. Boring locations on the cross section view are not at the correct startions, they are at the correct elevations.



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PROFILE — HORIZ: 1"=20' VERT: 1"=20'

DESIGN	BY Barrett Updegraff	CHECKED Benjamin Crawford
FIELD INVESTIGATION	BY Hailey Wagenman	

BRIDGE NO. 49C0037
POST MILES

PREPARED FOR THE
SAN LUIS OBISPO COUNTY
PUBLIC WORKS DEPARTMENT
976 OSOS STREET, ROOM 206, SAN LUIS OBISPO, CALIFORNIA 93408

DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT

LOG OF TEST BORINGS 2 OF 3

\\MAC\HOME\BOX\PROJECTS\17-375.1 DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT\CAD\LOTB 11X17\DWG\LOTB.DWG, 8/28/2018 2:58 PM, ----, BARRETTUPDEGRAFF

ELEVATION REFERENCE
Survey provided by Mark Thomas and Company, INC.
Dover Canyon_TO.dwg
Date: 6/22/18

Dist	COUNTY	ROAD NO.	JOB NO.	SHEET No.	TOTAL SHEETS
05	SLO			03	03

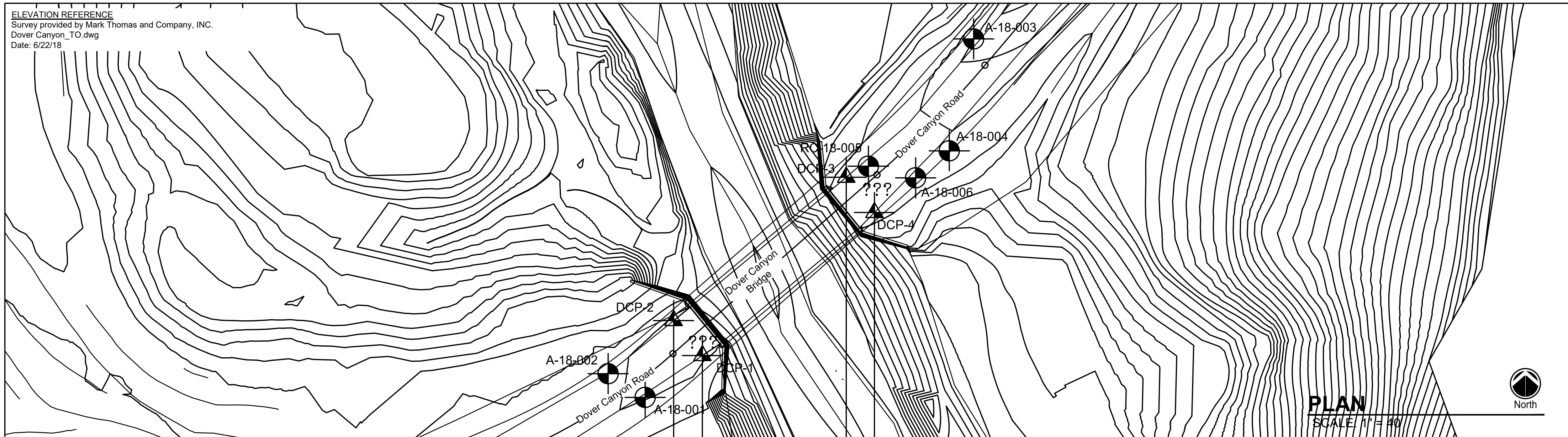
REGISTERED CIVIL ENGINEER — DATE 1/30/2020

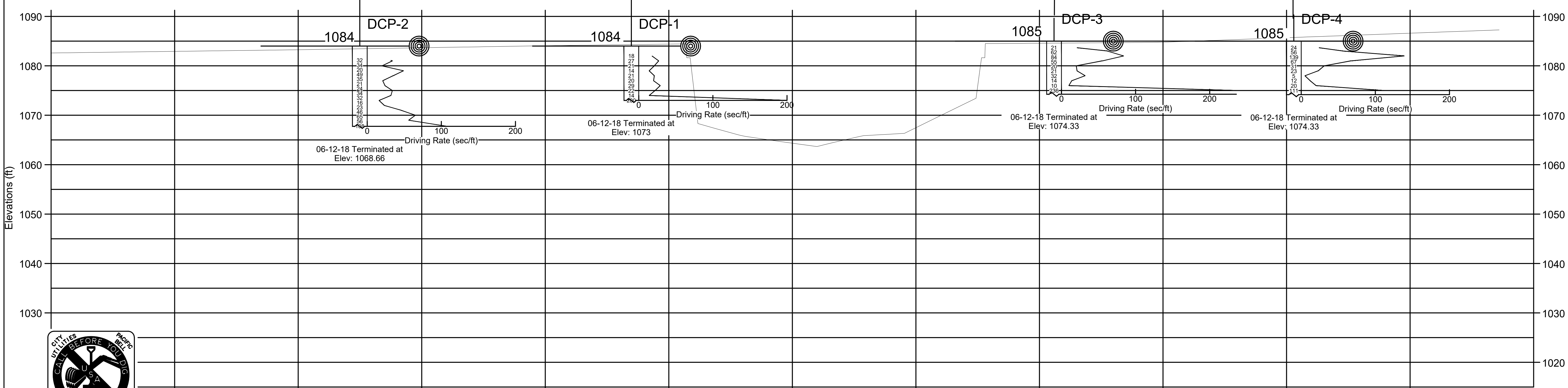
PLANS APPROVAL DATE 1/30/2020

REGISTERED PROFESSIONAL ENGINEER BENJAMIN D. CRAWFORD No. 68457 CIVIL STATE OF CALIFORNIA

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services
Taber Since 1954
1100 Corporate Way
Suite 230
Sacramento, CA 95831
(916) 455-4225

NOTES:
1. DCP tests were preformed with a WILDCAT P102/01 Dynamic Cone Penetrometer.
2. The DCP test consists of manually lifting and and dropping a 35lb hammer repeatedly to determine blow count per 10cm (~4in).
3. These tests were performed by Crawford and Associates, Inc. Project Engineers.
4. DCP locations on the cross section view are not at the correct stations, they are at the correct elevations.





CALL BEFORE YOU DIG
1-800-227-2600

PROFILE HORIZ: 1"=20' VERT: 1"=20'

DESIGN	BY Barrett Updegraff	CHECKED Benjamin Crawford	BRIDGE NO. 49C0037
FIELD INVESTIGATION	BY Hailey Wagenman		POST MILES

PREPARED FOR THE
SAN LUIS OBISPO COUNTY
PUBLIC WORKS DEPARTMENT
976 OSOS STREET, ROOM 206, SAN LUIS OBISPO, CALIFORNIA 93408

DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT

LOG OF TEST BORINGS 3 OF 3

\\MAC\HOME\BOX\PROJECTS\17-375.1 DOVER CANYON ROAD AT JACK CREEK BRIDGE REPLACEMENT PROJECT\CAD\LOTB 11X17\DWG\LOTB.DWG, 8/28/2018 2:58 PM, ----, BARRETTUPDEGRAFF

APPENDIX II

Dyanamic Cone Penetrometer Results

WILDCAT DYNAMIC CONE LOG

Crawford and Associates, Inc
1100 Corporate Way, Suite 230
Sacramento, CA, 95831

PROJECT NUMBER: 17-375.1
DATE STARTED: 06-12-2018
DATE COMPLETED: 06-12-2018

HOLE #: DCP-1
CREW: Hailey W., Amando and Kevin
PROJECT: Dover Canyon
ADDRESS: Dover Canyon Road
LOCATION: SLO County, USA

SURFACE ELEVATION: 1084 Ft
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	TESTED CONSISTENCY COHESIVE
-	-	-	-	-	-	-
-	-	-	-	-	-	-
- 1 ft	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
- 2 ft	7	31.1	•••••••••	8	LOOSE	MEDIUM STIFF
-	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
-	5	22.2	••••••	6	LOOSE	MEDIUM STIFF
- 3 ft	6	26.6	•••••••	7	LOOSE	MEDIUM STIFF
- 1 m	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	12	46.3	•••••••••••••	13	MEDIUM DENSE	STIFF
- 4 ft	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	3	11.6	•••	3	VERY LOOSE	SOFT
- 5 ft	4	15.4	••••	4	VERY LOOSE	SOFT
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	4	15.4	••••	4	VERY LOOSE	SOFT
- 6 ft	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 2 m	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 7 ft	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
- 8 ft	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	12	41.0	•••••••••••	11	MEDIUM DENSE	STIFF
- 9 ft	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
- 3 m 10 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	4	12.2	•••	3	VERY LOOSE	SOFT
-	4	12.2	•••	3	VERY LOOSE	SOFT
-	5	15.3	••••	4	VERY LOOSE	SOFT
- 11 ft	50/3"	153.0	••	25+	DENSE	HARD
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

C:\My Documents\Wildcat\WC_XL97.XLS

WILDCAT DYNAMIC CONE LOG

Crawford and Associates, Inc
1100 Corporate Way, Suite 230
Sacramento, CA, 95831

PROJECT NUMBER: 17-375.1
DATE STARTED: 06-12-2018
DATE COMPLETED: 06-12-2018

HOLE #: DCP-2
CREW: Hailey W., Amando and Kevin
PROJECT: Dover Canyon
ADDRESS: Dover Canyon Road
LOCATION: SLO County, USA

SURFACE ELEVATION: 1084 Ft
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

	BLOWS	RESISTANCE	GRAPH OF CONE RESISTANCE		TESTED CONSISTENCY	
DEPTH	PER 10 cm	Kg/cm²	0 50 100 150	N'	NON-COHESIVE	COHESIVE
-	-	-	-	-	-	-
-	-	-	-	-	-	-
- 1 ft	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
- 2 ft	2/ last 2"	8.9	••	2	VERY LOOSE	SOFT
-	12	53.3	•••••••••••••••	15	MEDIUM DENSE	STIFF
-	8	35.5	••••••••••	10	LOOSE	STIFF
- 3 ft	10	44.4	••••••••••••	12	MEDIUM DENSE	STIFF
- 1 m	13	57.7	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	11	42.5	••••••••••••	12	MEDIUM DENSE	STIFF
- 4 ft	5	19.3	•••••	5	LOOSE	MEDIUM STIFF
-	4	15.4	••••	4	VERY LOOSE	SOFT
-	11	42.5	••••••••••••	12	MEDIUM DENSE	STIFF
- 5 ft	15	57.9	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	18	69.5	•••••••••••••••••••	19	MEDIUM DENSE	VERY STIFF
-	16	61.8	•••••••••••••••••	17	MEDIUM DENSE	VERY STIFF
- 6 ft	13	50.2	••••••••••••••	14	MEDIUM DENSE	STIFF
-	12	46.3	•••••••••••••	13	MEDIUM DENSE	STIFF
- 2 m	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
- 7 ft	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
- 8 ft	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
- 9 ft	10	34.2	•••••••••	9	LOOSE	STIFF
-	12	41.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	12	41.0	•••••••••••	11	MEDIUM DENSE	STIFF
- 3 m 10 ft	12	41.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	12	36.7	••••••••••	10	LOOSE	STIFF
-	8	24.5	••••••	6	LOOSE	MEDIUM STIFF
-	5	15.3	••••	4	VERY LOOSE	SOFT
- 11 ft	5	15.3	••••	4	VERY LOOSE	SOFT
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
- 12 ft	6	18.4	•••••	5	LOOSE	MEDIUM STIFF
-	11	33.7	•••••••••	9	LOOSE	STIFF
-	17	52.0	••••••••••••••	14	MEDIUM DENSE	STIFF
- 4 m 13 ft	16	49.0	•••••••••••••	13	MEDIUM DENSE	STIFF
	13	36.0	••••••••••	10	LOOSE	STIFF

WILDCAT DYNAMIC CONE LOG

PROJECT: Dover Canyon PROJECT NUMBER: 17-375.1

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	TESTED CONSISTENCY COHESIVE
-	13	36.0	••••••••••	10	LOOSE	STIFF
-	14	38.8	•••••••••••	11	MEDIUM DENSE	STIFF
- 14 ft	30	83.1	•••••••••••••••••••••••	23	MEDIUM DENSE	VERY STIFF
-	21	58.2	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
-	14	38.8	•••••••••••	11	MEDIUM DENSE	STIFF
- 15 ft	10	27.7	••••••••	7	LOOSE	MEDIUM STIFF
-	32	88.6	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
-	50/3"	138.5	••••••••••••••••••••••••••••••••••••••	25+	DENSE	HARD
- 16 ft						
- 5 m						
-						
- 17 ft						
-						
-						
- 18 ft						
-						
-						
- 19 ft						
-						
- 6 m						
- 20 ft						
-						
-						
- 21 ft						
-						
-						
- 22 ft						
-						
-						
- 7 m 23 ft						
-						
-						
- 24 ft						
-						
-						
- 25 ft						
-						
-						
- 26 ft						
- 8 m						
-						
- 27 ft						
-						
-						
- 28 ft						
-						
-						
- 29 ft						
-						
- 9 m						

WILDCAT DYNAMIC CONE LOG

Crawford and Associates, Inc
1100 Corporate Way, Suite 230
Sacramento, CA, 95831

PROJECT NUMBER: 17-375.1
DATE STARTED: 06-12-2018
DATE COMPLETED: 06-12-2018

HOLE #: DCP-3
CREW: Hailey W., Amando and Kevin
PROJECT: Dover Canyon
ADDRESS: Dover Canyon Road
LOCATION: SLO County, USA

SURFACE ELEVATION: 1085 Ft
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	COHESIVE
-	50/4"	222.0	••	25+	VERY DENSE	HARD
-	-	-	-	-	-	-
- 1 ft	-	-	-	-	-	-
-	4	17.8	•••••	5	LOOSE	MEDIUM STIFF
-	17	75.5	•••••••••••••••••••••	21	MEDIUM DENSE	VERY STIFF
- 2 ft	24	106.6	••••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	18	79.9	••••••••••••••••••••••	22	MEDIUM DENSE	VERY STIFF
-	20	88.8	•••••••••••••••••••••••••	25	MEDIUM DENSE	VERY STIFF
- 3 ft	27	119.9	••••••••••••••••••••••••••••••••••	25+	DENSE	HARD
- 1 m	31	137.6	•••••••••••••••••••••••••••••••••••••••	25+	DENSE	HARD
-	26	100.4	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 4 ft	24	92.6	••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	19	73.3	••••••••••••••••••••	20	MEDIUM DENSE	VERY STIFF
-	12	46.3	•••••••••••••	13	MEDIUM DENSE	STIFF
- 5 ft	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	6	23.2	••••••	6	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 6 ft	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
-	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 2 m	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 7 ft	11	37.6	••••••••••	10	LOOSE	STIFF
-	12	41.0	•••••••••••	11	MEDIUM DENSE	STIFF
-	9	30.8	••••••••	8	LOOSE	MEDIUM STIFF
- 8 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	4	13.7	•••	3	VERY LOOSE	SOFT
-	4	13.7	•••	3	VERY LOOSE	SOFT
- 9 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
- 3 m 10 ft	4	13.7	•••	3	VERY LOOSE	SOFT
-	32	97.9	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	50/1"	153.0	•••	25+	DENSE	HARD
-						
- 11 ft						
-						
-						
- 12 ft						
-						
-						
- 4 m 13 ft						

WILDCAT DYNAMIC CONE LOG

Crawford and Associates, Inc
1100 Corporate Way, Suite 230
Sacramento, CA, 95831

PROJECT NUMBER: 17-375.1
DATE STARTED: 06-12-2018
DATE COMPLETED: 06-12-2018

HOLE #: DCP-4
CREW: Hailey W., Amando and Kevin
PROJECT: Dover Canyon
ADDRESS: Dover Canyon Road
LOCATION: SLO County, USA

SURFACE ELEVATION: 1085 Ft
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY NON-COHESIVE	TESTED CONSISTENCY COHESIVE
-	-	-	-	-	-	-
-	-	-	-	-	-	-
- 1 ft	-	-	-	-	-	-
-	-	-	-	-	-	-
-	24	106.6	••••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
- 2 ft	25	111.0	••••••••••••••••••••••••••••••••	25+	DENSE	HARD
-	22	97.7	••••••••••••••••••••••••••••	25+	MEDIUM DENSE	VERY STIFF
-	9	40.0	•••••••••••	11	MEDIUM DENSE	STIFF
- 3 ft	45	199.8	•••	25+	VERY DENSE	HARD
- 1 m	50/3" *	222.0	••	25+	VERY DENSE	HARD
-	44	169.8	•••	25+	DENSE	HARD
- 4 ft	30	115.8	•••••••••••••••••••••••••••••••••	25+	DENSE	HARD
-	22	84.9	••••••••••••••••••••••••	24	MEDIUM DENSE	VERY STIFF
-	15	57.9	••••••••••••••••	16	MEDIUM DENSE	VERY STIFF
- 5 ft	13	50.2	••••••••••••••	14	MEDIUM DENSE	STIFF
-	10	38.6	•••••••••••	11	MEDIUM DENSE	STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 6 ft	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
-	8	30.9	••••••••	8	LOOSE	MEDIUM STIFF
- 2 m	7	27.0	•••••••	7	LOOSE	MEDIUM STIFF
- 7 ft	5 **	17.1	••••	4	VERY LOOSE	SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
-	0	0.0		0	VERY LOOSE	VERY SOFT
- 8 ft	1	3.4		0	VERY LOOSE	VERY SOFT
-	3	10.3	••	2	VERY LOOSE	SOFT
-	8	27.4	•••••••	7	LOOSE	MEDIUM STIFF
- 9 ft	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
-	7	23.9	••••••	6	LOOSE	MEDIUM STIFF
-	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
- 3 m 10 ft	6	20.5	•••••	5	LOOSE	MEDIUM STIFF
-	5	15.3	••••	4	VERY LOOSE	SOFT
-	50/2"	153.0	••	25+	DENSE	HARD
-						
- 11 ft						
-	Notes: * At 1 m a 50/3" blow count was recorded, the soil was hand augered and a coarse gravel in the way was removed					
-	'** At 7 feet the last of the 5 blows went all the way to 8 feet					
- 12 ft						
-						
-						
- 4 m 13 ft						

APPENDIX III

Laboratory Test Results

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 5/25/18
Technician: HFW/AC

MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-002-1A	A-18-003-1A	A-18-003-3A	A-18-003-4A	A-18-004-1A
USCS Symbol	ML	ML	CH	CL	CL
Depth (ft.)	3.5	3.5	11	16	3.5
Sample Length (in.)	3.083	5.270	5.983	5.755	4.869
Diameter (in.)	2.390	2.376	2.371	2.375	2.390
Sample Volume (ft^3)	0.00800	0.01352	0.01529	0.01476	0.01264
Total Mass Soil+Tube (g)	676.4	966.2	779.4	712.0	782.0
Mass of Tube (g)	272.2	250.8	0.0	0.0	276.1
Tare No.	A2	D4	G25	R10	A19
Tare (g)	13.7	13.7	20.7	131.2	20.8
Wet Soil + Tare (g)	76.0	73.2	64.2	382.9	61.1
Dry Soil + Tare (g)	66.4	62.7	52.0	308.1	55.8
Dry Soil (g)	52.7	49.0	31.3	176.9	35.0
Water (g)	9.7	10.5	12.2	74.8	5.4
Moisture (%)	**18.3**	**21.4**	**38.9**	**42.3**	**15.3**
Dry Density (pcf)	**94.1**	**96.1**	**80.9**	**74.8**	**76.5**

Notes:

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 5/25/18
Technician: HFW/AC

MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-004-2A	A-18-004-3A			
USCS Symbol	CL	CL			
Depth (ft.)	6	11			
Sample Length (in.)	5.326	5.788			
Diameter (in.)	2.379	2.382			
Sample Volume (ft^3)	0.01370	0.01493			
Total Mass Soil+Tube (g)	693.3	713.1			
Mass of Tube (g)	0.0	0.0			
Tare No.	G8	R7			
Tare (g)	13.2	130.0			
Wet Soil + Tare (g)	75.6	368.9			
Dry Soil + Tare (g)	64.4	313.9			
Dry Soil (g)	51.2	183.9			
Water (g)	11.2	55.0			
Moisture (%)	**21.8**	**29.9**			
Dry Density (pcf)	**91.6**	**81.1**			

Notes:

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 5/25/18
Technician: HFW/AC

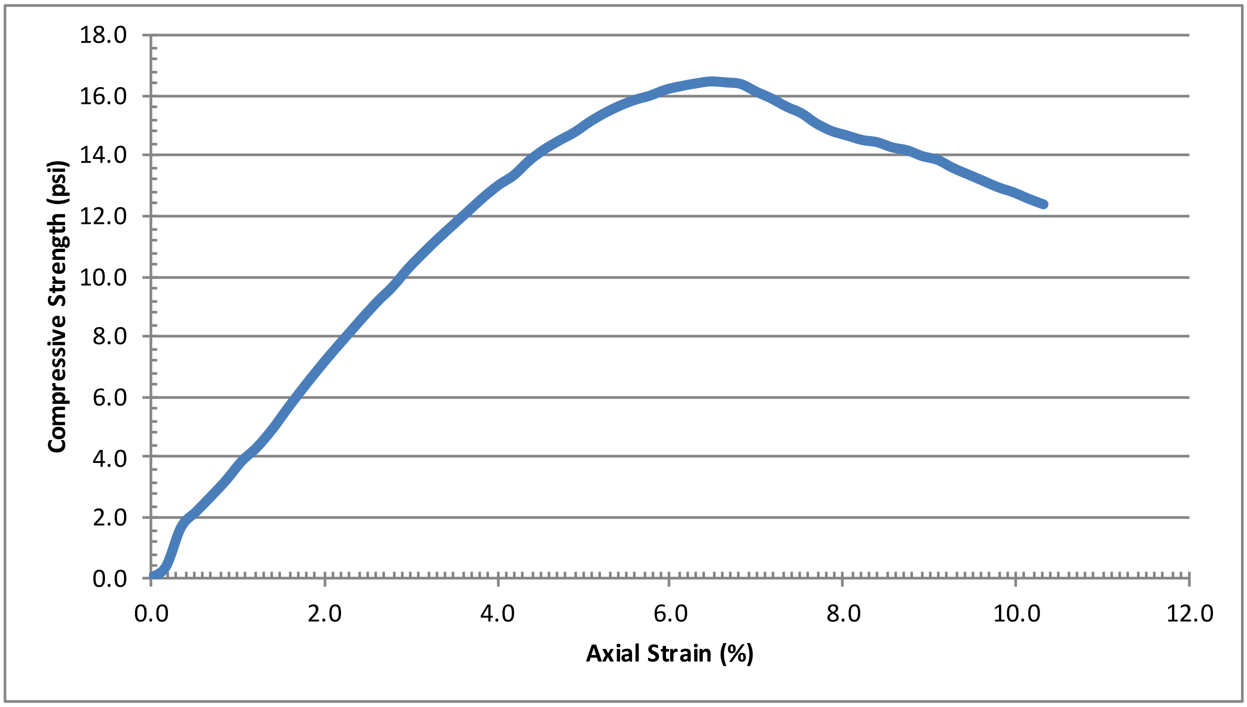
Pocket Pen Results

Boring ID	Depth (ft)	Pocket Pen Value
A-18-002-1A	3.5	2.00
A-18-003-1A	3.5	3.00
A-18-003-3A	11.0	1.00
A-18-003-4A	16.0	1.50
A-18-004-2A	6.0	3.75
A-18-004-3A	11.0	+4.50

Notes:

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 7/25718
Technician: HFW
Sample ID: A-18-003-4A Depth (ft): 16.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

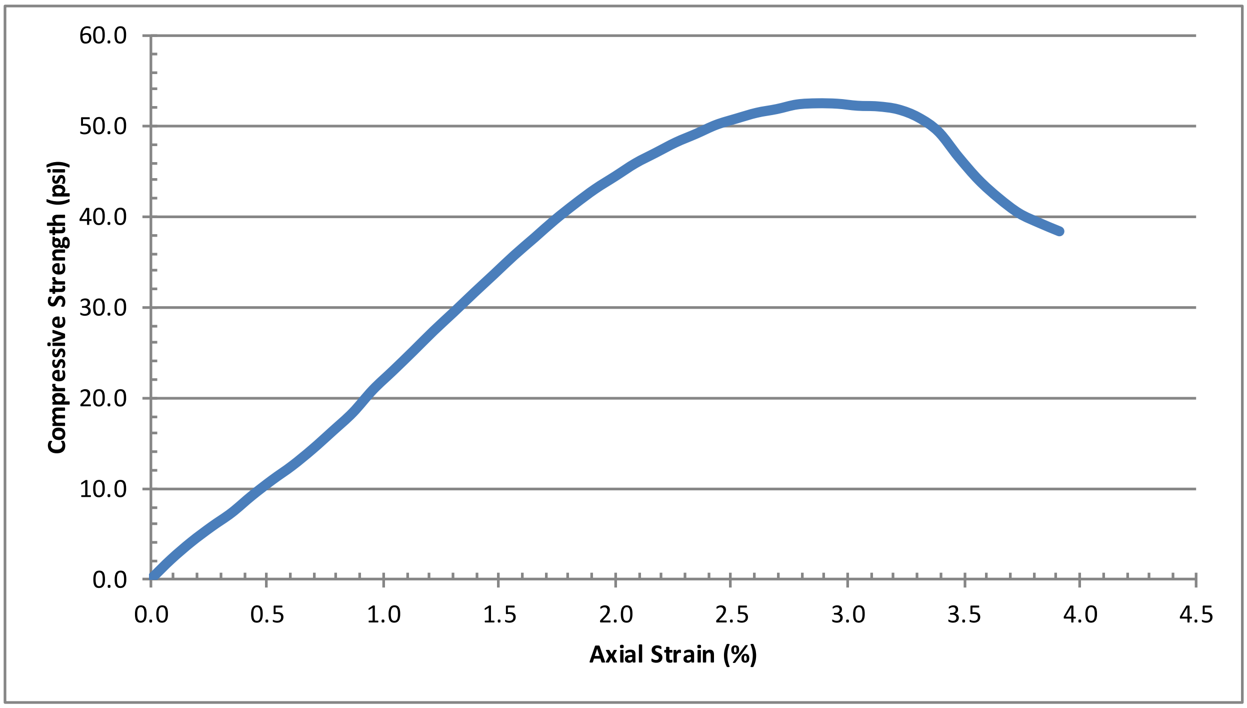


Dry Density (pcf)	**74.7**
Water Content (%)	**42.3**
Unconfined Compressive Strength (psi)	**16.5**
Unconfined Compressive Strength (psf)	**2376**
Shear Strength (psf)	**1188**
Average Height (in)	5.755
Average Diameter (in)	2.375
Rate of strain (%)	1.0
Strain at Failure (%)	6.5

Notes:

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 7/25718
Technician: HFW
Sample ID: A-18-004-3A Depth (ft): 11.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166



Dry Density (pcf)	**81.1**
Water Content (%)	**29.9**
Unconfined Compressive Strength (psi)	**52.5**
Unconfined Compressive Strength (psf)	**7560**
Shear Strength (psf)	**3780**
Average Height (in)	5.788
Average Diameter (in)	2.382
Rate of strain (%)	0.5
Strain at Failure (%)	2.9

Notes:

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 7/25/18
Technician: HFW

200 Wash - ASTM D1140
Method A

Max Particle Size (100% Passing)	Standard Sieve Size	Recommended Min Mass of Test Specimens
2 mm or less	No. 10	20 g
4.75 mm	No. 4	100 g
9.5 mm	3/8 "	500 g
19.0 mm	3/4 "	2.5 kg
37.5 mm	1 1/2 "	10 kg
75.0 mm	3 "	50 kg

Table from 6.2 of ASTM D1140

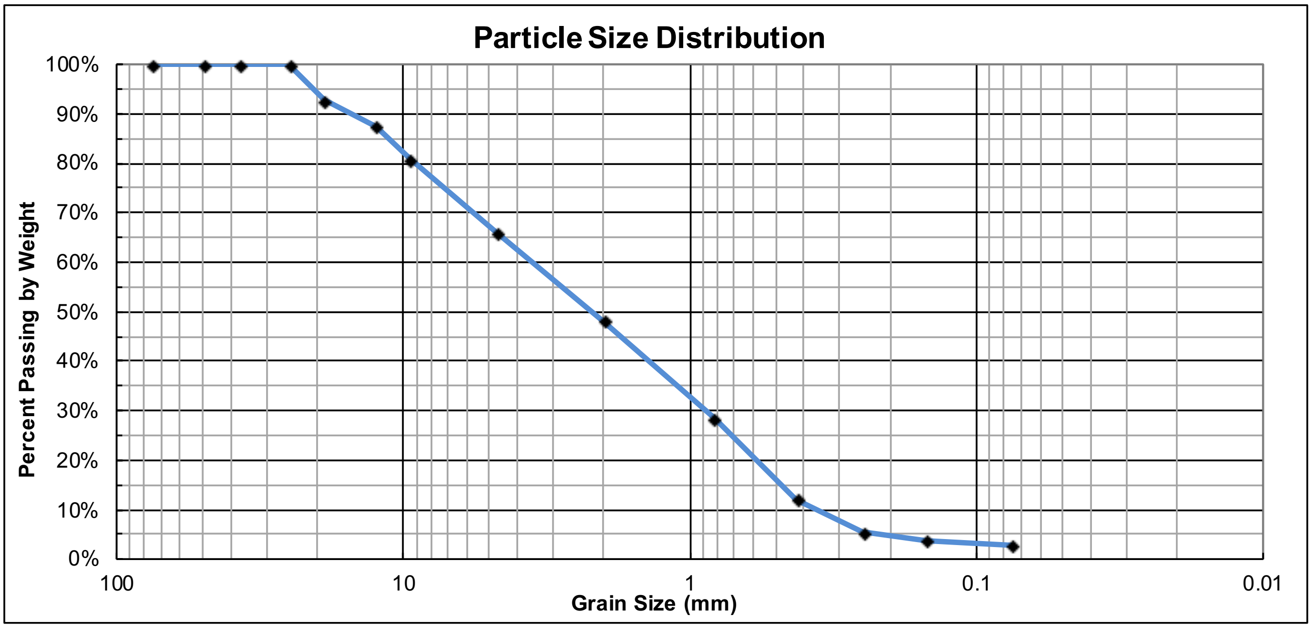
Sample No.	A-18-002-3A	A-18-003-4A	A-18-004-3A		
USCS Symbol	SP-SM	CL	CL		
Depth (ft.)	10.5	16	11		
Tare No.	P7	R10	R7		
Tare (g)	131.9	131.2	130		
Dry Soil + Tare (g)	341.2	308.1	313.9		
Dry Mass before (g)	209.3	176.9	183.9		
Dry Mass after (g)	186.5	82.4	59.2		
Percent Fines (%)	11	53	68		

Notes:

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement Project
CAInc File No: 17-375.1
Date: 5/9/18
Technician: AC
Sample ID: Bulk Sample from Channel Bank
Depth (ft): -
USCS Classification: Poorly Graded Sand with Gravel (SP)

ASTM 6913 - Method A



% Cobble	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
	7	27	18	36	9	
0	**34**		**63**			**3**

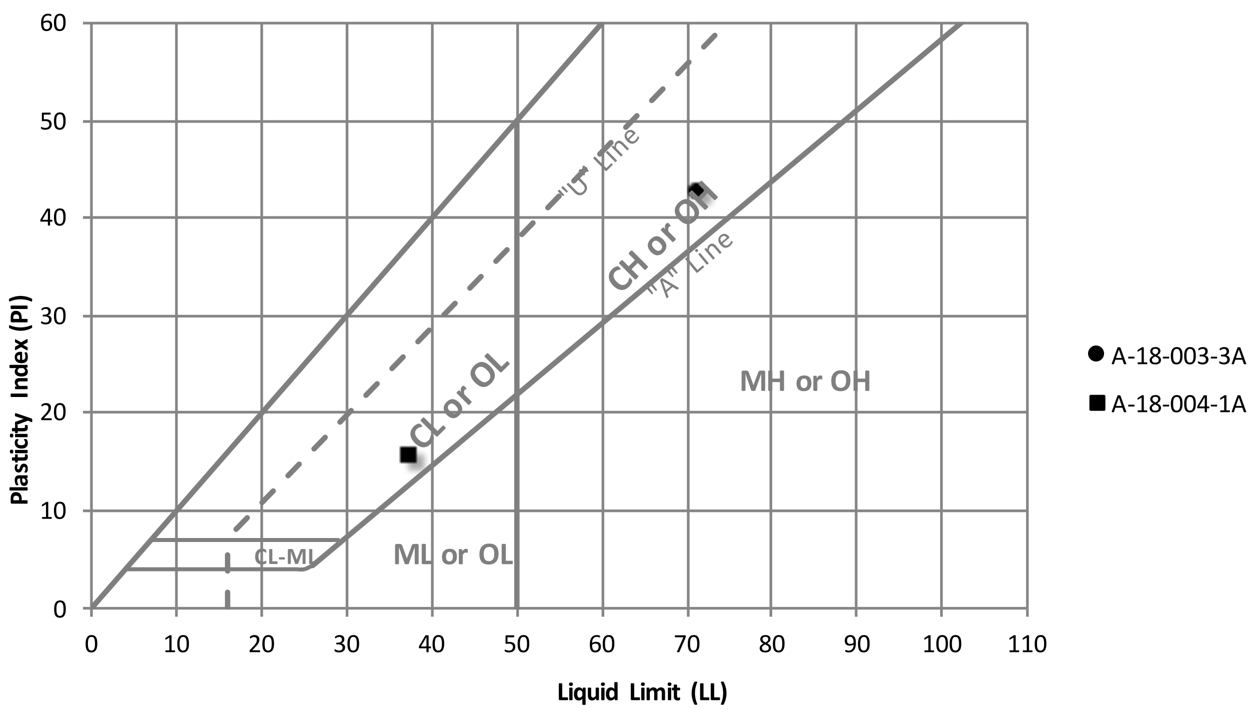
		Sieve #	Opening mm	Cummulative Mass Retained (g)	% Passing %
Cobbles		**3"**	75	0.0	100%
Gravel	Coarse	**2"**	50	0.0	100%
		1-1/2"	37.5	0.0	100%
		1"	25.0	0.0	100%
		3/4"	19.0	19.4	93%
	Fine	**1/2"**	12.5	33.8	87%
		3/8"	9.50	51.8	81%
		#4	4.75	91.4	66%
Sand	Coarse	**#10**	2.00	139.6	48%
	Medium	**#20**	0.825	192.2	29%
		#40	0.425	236.3	12%
	Fine	**#60**	0.250	254.3	5%
		#100	0.150	258.9	4%
		#200	0.075	261.5	3%

Coefficient of Uniformity	Coefficient of Curvature	50% of Cumulative Mass
Cu = 10.4	Cc = 0.6	D_{50} = 2.2

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 6/26/18
Technician: GL

Plastic Index - ASTM D4318

Sample ID	Depth (ft)	Liquid Limit	Plastic Limit	PI
A-18-003-3A	11	71	28	43
A-18-004-1A	3.5	37	21	16



Plasticity Chart

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Dover Canyon Road at Jack Creek Bridge Replacement
CAInc File No: 17-375.1
Date: 8/21/18-8/22/18
Technician: GL/HFW

Boring	Top Hole Elev. (feet)	Core Run	Depth (feet)	Elev. (feet)	Core Diameter (inches)	Failure Load (P) (lbf)	Point Load Index (I_S) (psi)	Correlated Uniaxial Compressive Strength (psi)	Remarks/Notes
RC-18-005	1085	**E**	37.1	1047.9	2.39	145	25	620	Vertical Shear
RC-18-005	1085	**F**	42.2	1042.8	2.39	294.94	52	1200	Irregular Shear
RC-18-005	1085	**G**	45.3	1039.7	2.39	177.56	31	700	Approximately 60 Degree Angle Shear
RC-18-005	1085	**I**	55.4	1029.6	2.39	98.24	17	400	Approximately 60 Degree Angle Shear
RC-18-005	1085	**K**	62.2	1022.8	2.39	177.82	31	700	Vertical Shear

Uniaxial compressive strength values based on point load test data and correlations derived from Bieniawski (1975); "Rock Mechanics for Underground Mining", Brady & Brown, 1985 (page 98-99).

Equation to Determine Uniaxial Compressive Strength:

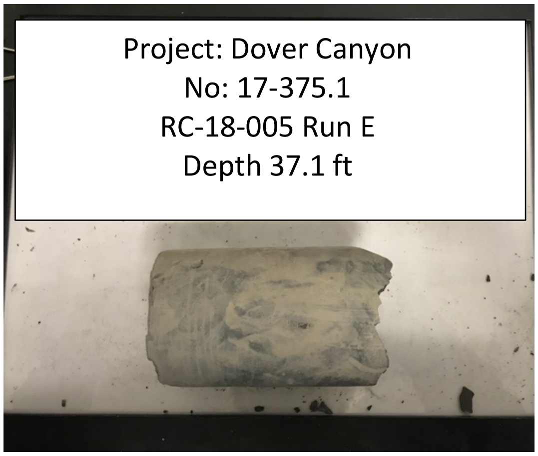
Uniaxial Compressive Strength = $\sigma_c = (14 + 0.175D)I_s$

Point Load Index = $I_s = P/D^2$

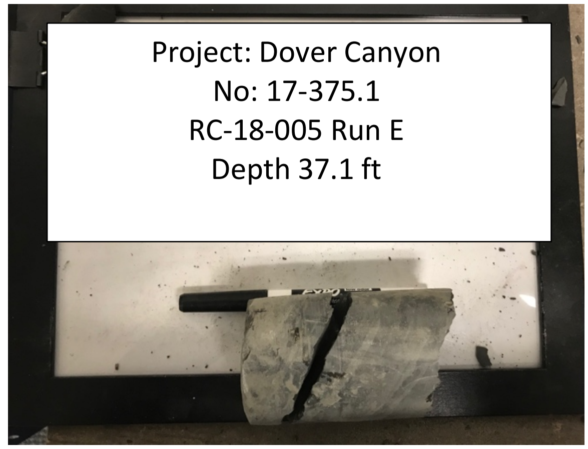
Point Load Photo Log: 17-375.1: Dover Canyon Road at Jack Creek Bridge Replacement

<u>RC-18-005 Run E at 37.1 ft bgs</u>

Before:



After:



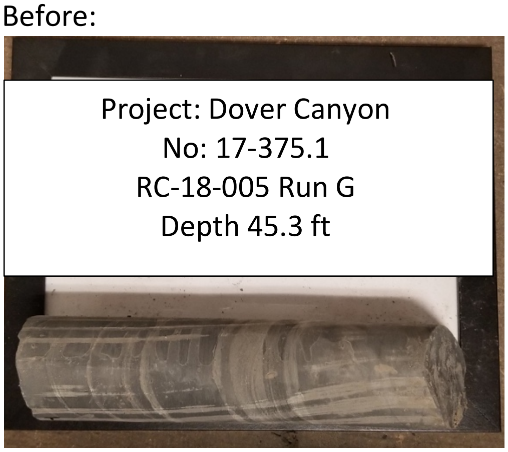
<u>RC-18-005 Run F at 42.2 ft bgs</u>

Before:

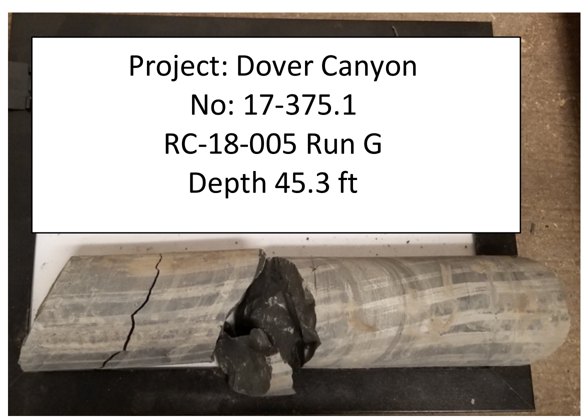
After:

<u>RC-18-005 Run G at 45.3 ft bgs</u>

Before:



After:

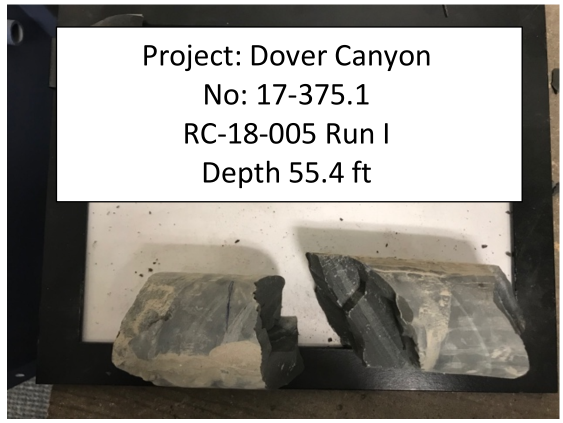


Point Load Photo Log: 17-375.1: Dover Canyon Road at Jack Creek Bridge Replacement

RC-18-005 Run I at 55.4 ft bgs

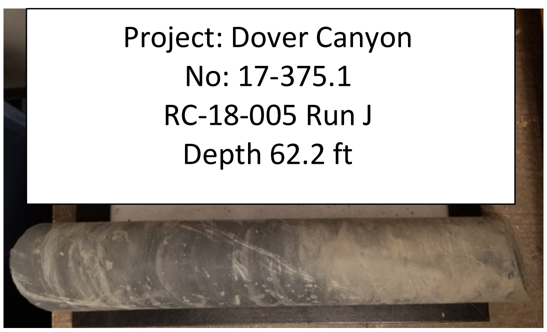
Before:

After:

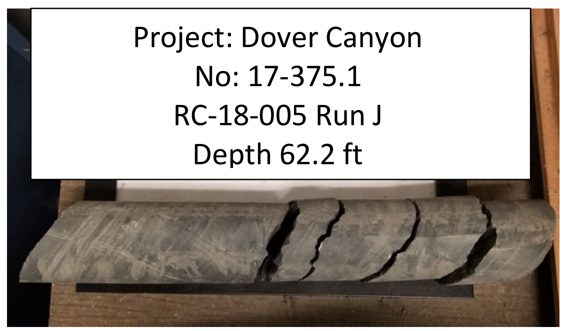


RC-18-005 Run J at 62.2 ft bgs

Before:



After:



Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/29/2018
Date Submitted 06/26/2018

To: Hailey Wagenman
Crawford & Associates, Inc.
1100 Corporate Way STE. 230
Sacramento, CA 95831-6120

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

The reported analysis was requested for the following location:
Location : 17-375.1 Site ID : A-18-003-1A.
Thank you for your business.

* For future reference to this analysis please use SUN # 77363-161497.

EVALUATION FOR SOIL CORROSION

Soil pH	5.65	
Minimum Resistivity	2.28 ohm-cm (x1000)	
Chloride	1.5 ppm	00.00015 %
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
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/29/2018
Date Submitted 06/26/2018

To: Hailey Wagenman
Crawford & Associates, Inc.
1100 Corporate Way STE. 230
Sacramento, CA 95831-6120

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

The reported analysis was requested for the following location:
Location : 17-375.1 Site ID : A-18-003-4A.
Thank you for your business.

* For future reference to this analysis please use SUN # 77363-161498.

EVALUATION FOR SOIL CORROSION

Soil pH	6.77	
Minimum Resistivity	0.62 ohm-cm (x1000)	
Chloride	18.6 ppm	00.00186 %
Sulfate	178.1 ppm	00.01781 %

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