REVISED FINAL FOUNDATION REPORT

Replacement of Robinson Creek Bridge at Lambert Lane

Boonville, Mendocino County, California
Crawford File No. 16-278.1
Bridge No.: 10C-0146
BRLO No. 5910 (099)

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January 21, 2022 (Revised July 6, 2022)

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Mr. Jim L. Foster, Jr., P.E.
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REVISED FINAL FOUNDATION REPORT
Replacement of Robinson Creek Bridge at Lambert Lane
Bridge No. 10C-0146
Federal Project No. BRLO-5910 (099)
Boonville, Mendocino County, CA

Dear Mr. Foster,

Crawford & Associates, Inc. (Crawford) prepared this Revised Final Foundation Report for the Replacement of Robinson Creek Bridge at Lambert Lane project in Mendocino County, California. Crawford prepared this report in accordance with our May 10, 2016 agreement between Crawford and Quincy Engineering, Inc. (QEI). This report supersedes all previous versions of this report and incorporates revisions based on review comments received from Caltrans.

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.,

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1 INTRODUCTION

1.1 PURPOSE

Crawford & Associates, Inc. (Crawford) prepared this Revised Final Foundation Report for the Replacement of Robinson Creek Bridge at Lambert Lane in Mendocino County, California in accordance with our May 10, 2016 agreement between Crawford and Quincy Engineering, Inc. (QEI). The purpose of this report is to provide earth materials criteria for use in the design of the proposed new bridge foundations and retaining wall structure. It includes the results of the subsurface exploration, laboratory testing results, the Log of Test Borings (LOTB), and foundation recommendations for the proposed bridge and retaining wall structure.

This report supersedes all previous versions of this report and incorporates revisions based on review comments received from Caltrans. A copy of the Caltrans review comments and our responses are included as Appendix VI.

QEI indicates that bridge design is with respect to Caltrans 2018 Standard Specifications and Standard Plans with the 2017 American Association of State Highway and Transportation Officials Load and Resistance Factor – Bridge Design Specifications (AASHTO LRFD – BDS), 8th Edition and Caltrans Amendments.

1.2 GEOTECHNICAL SERVICES

To prepare this report, Crawford:

- discussed the project with Quincy Engineering, Inc. (QEI);
- attended a site kickoff meeting on May 11, 2016;
- reviewed Bridge Inspection Records Information System (BIRIS) report for the existing bridge dated May 5, 2014;
- reviewed available published geologic and seismic mapping pertaining to the site;
- reviewed As-built plans for the existing bridge;
- reviewed Planning Study drawings and other preliminary documents prepared by QEI;
- completed a site review on August 26, 2015;
- completed an Initial Site Assessment (ISA) dated July 5, 2016;
- published a Preliminary Foundation Report, dated November 29, 2016;
- completed a second site review on February 2, 2018;
- completed subsurface exploration at the site on April 17-19, 2018;
- completed laboratory testing on soil samples obtained during subsurface exploration;
- reviewed preliminary 65% project plans with mark-ups provided by QEI on October 19, 2021;
- published a Draft Foundation Report, dated October 29, 2021; and
- performed geotechnical engineering evaluation and analysis.

This report supersedes our Draft Foundation Report, dated October 29, 2021, for Robinson Creek Bridge. Limitations of this study are discussed in the final section of this report.

2 PROJECT DESCRIPTION

The project site is in Boonville, Mendocino County, California. Boonville is situated at the southeast end of Anderson Valley along SR 128, approximately 0.5 miles northwest of the intersection of SR 128 and SR 258, and 13 miles southwest of Ukiah. The bridge location is within Section 2 of T13N, R14W and is located approximately 0.1 miles southwest SR 128 on Lambert Lane (CR 123-A) over Robinson Creek. Site coordinates are approximately latitude 39.0084°N and longitude 123.3683°W. Refer to Figure 1 for the Vicinity Map in Appendix I. All elevations referenced within this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted.

The proposed project will replace the existing bridge on the existing alignment with staged construction. The new bridge will be a single-span structure that is 96.0 feet long and 31.5 feet measured on the “L Line” in QEI’s “General Plan”. The longer span compared to the existing bridge will allow the abutments to move back from the creek bed and be protected from scour. The bridge superstructure is shown as a steel plate girder.

The new abutments will be located behind the existing abutments (outside the creek channel) and skewed 30 degrees to match the channel alignment. New bridge abutments are shown at Sta. 4+92.00 (BB, elev. 390.00) and Sta. 5+88.00 (EB, elev. 390.66). The end-slopes in front of the bridge abutments are shown at 1.5:1 (horizontal:vertical) with rock slope protection (RSP).

Some channel modification is anticipated to reduce the sharp, 90-degree bends and smooth the channel geometry to a modified “S” curve. The channel modifications will also consider improvements to fish passage and riparian habitat.

The new substructure will consist of seat-type, wall abutments with cantilevered wingwalls parallel to Lambert Lane. The new structure foundations are planned to be 30-inch diameter cast-in-drilled-hole (CIDH) piles with 9 piles per support. A nominal resistance (Strength/Construction Limit State) of about 400 kips/pile has been indicated for the 30-inch CIDH piles at Abutment 1 and 2. No tension demand is indicated for new abutment piles.

A 6 ft high Caltrans Standard Retaining Wall Type 1A (Case 1) is also planned along the southerly edge of roadway (eastbound lane) that extends from the end of the wingwall at Abutment 1 to the southwest. The Retaining Wall is shown to be 28 ft long. The retaining wall is proposed to be supported on spread footing foundations at an approximate elev. 382.83 ft. New approach roadway improvements will also be completed at each abutment of the proposed bridge. The general plan shows the bridge profile grade will be raised slightly (approximately less than 1 ft). Structure approach slabs are also shown at each abutment.

3 GEOTECHNICAL INVESTIGATION

Field investigation for replacement of the Robinson Creek Bridge at Lambert Lane consisted of three exploratory borings. Crawford retained Taber Drilling to complete the borings from April 17 through 19, 2019. Taber Drilling used a CME 55 track rig to drill the borings with 4-inch hollow-stem auger and 94-mm mud rotary drilling equipment. The maximum depth of exploration was 111.0±ft below the ground surface (bgs) (R-18-002; lowest elev. 277.5±ft).

3.1 GEOTECHNICAL TEST BORINGS

3.1.1 DRILLING AND SAMPLING

A summary of the subsurface exploration is provided in Table 1. The boring locations are shown on Figure 2 in Appendix I and on the Log of Test Borings (LOTB) in Appendix II.

Table 1: Subsurface Exploration Summary

Support Location	Boring Designation	Completion Date	Drill Rig Type	Hammer Type	Hammer Efficiency (%)	Approx. Ground Surface Elevation (feet)	Boring Depth (feet)
Abutment 1	R-18-002	4/18/2018	CME 55	Automatic	82	388.5	111.0
Abutment 1 (Retaining Wall)	R-18-003	4/19/2018	CME 55	Automatic	82	389.5	61.0
Abutment 2	R-18-001	4/17/2018	CME 55	Automatic	82	388.3	91.0

Soil samples were recovered by means of 2.0-inch O.D. Standard Penetration Test (SPT) split-spoon sampler (ASTM D1586) with 1.4-inch I.D. stainless steel liners and a 3.0-inch O.D. "California" split-spoon sampler (ASTM D3550) with 2.4-inch I.D. stainless steel liners. The samplers were advanced with standard 350 ft-lb striking force using a 140-lb automatic hammer and a drop height of 30 inches. A hammer energy calibration test was not performed specifically for this project/site. At the time of the field exploration, Taber Drilling reported the last energy calibration performed on the hammer used in the field for this project has an average efficiency of 82%.

Drive samples taken in the borings were typically collected at approximate 5 ft intervals and as otherwise directed by the field engineer/geologist. At each test interval, the sampler was driven 18 inches (or until sampler refusal criterion was met), and the blows necessary to advance the sampler each 6 inches of penetration were recorded. The sample refusal criterion is defined as 50 or more blows with less than 6 inches of sampler advancement.

Bulk samples of soil cuttings were also collected at selected depths from the borings.

3.1.2 LOGGING

Crawford's field personnel logged the test borings consistent with the Unified Soil Classification System and the Caltrans Logging Manual[1]. Selected portions of recovered soil samples from the driven samplers were retained in sealed containers for laboratory testing and reference. The bulk soil samples collected from the auger cuttings were placed in plastic bags for laboratory testing and reference. Groundwater observations were recorded during and after drilling operations. While logging in the field, Crawford also completed pocket penetrometer tests on drive samples of cohesive soils to determine consistency and provide an estimate of unconfined compressive strength.

[1] Caltrans, Soil and Rock Logging, Classification, and Presentation Manual, 2010 Edition

3.1.3 SAMPLER PENETRATION RESISTANCE (N-VALUE)

The in-situ sampler penetration resistance (N-value) in blows per foot was recorded in the field to obtain an approximate measure of the dynamic resistance of the soil. The N-value was recorded as the number of hammer blows necessary to drive the sampler the final 12-inches of the 18-inch sample interval, unless refusal was met.

The SPT N-value adjusted to 60% hammer energy (N_{60}) is routinely used to provide an index of the apparent density of cohesionless soils and sometimes (albeit less reliably) to estimate the consistency of cohesive soils. The energy-corrected N_{60} value normalized for effective overburden stress referred to as $(N_1)_{60}$ is typically used to correlate soil strength parameters and bearing characteristics.

For a non-standard sampler (i.e., non-SPT sampler), the in-situ N-value was corrected to an Equivalent SPT N-Value using values published in Caltrans Geotechnical Manual (March 2021), then adjusted to provide an *Equivalent* N_{60} and/or *Equivalent* $(N_1)_{60}$ value that can be correlated to soil strength and bearing characteristics for use in geotechnical analysis.

The in-situ (uncorrected) N-values are shown on the LOTB drawing provided in Appendix II.

3.1.4 BOREHOLE ABANDONMENT

At completion, the exploratory borings were backfilled with cement grout in accordance with the county boring permit requirements and sealed with asphalt cold patch at the surface.

3.1.5 BORING LOCATIONS AND ELEVATIONS

The boring locations were measured in the field with respect to existing site features and then referenced to project stationing. Crawford also surveyed the boring location holes to known elevations for the general plan provided by QEI. The details and locations of test borings are shown on the LOTB drawing (Appendix II) and on Figure 2 (Appendix I). Manny Gutierrez was Crawford's field engineer for this study.

4 LABORATORY TESTING PROGRAM

Laboratory tests have been completed on selected representative samples obtained from the exploratory borings to aid in soil classification and evaluate the physical and engineering properties of the earth materials for use in geotechnical analysis required for the project such as liquefaction potential, shallow and deep foundations, and corrosion potential.

Crawford completed the following laboratory tests on representative soil samples from the borings:

- Atterberg Limits (ASTM D4318)
- Corrosivity Testing (CTM 643, CTM 417, and CTM 422)
- Moisture Content/Unit Weight (ASTM D2216/2937)
- No. 200 Sieve (ASTM D1140)
- Particle Size Distribution (ASTM D6913)
- R-value (CTM 301)
- Unconfined Compression (ASTM D2166)

Laboratory tests were used for soil classification, corrosivity, and strength parameters. Laboratory test results are provided in Appendix III and the exploratory boring locations are shown on the LOTB included in Appendix II.

5 GEOTECHNICAL CONDITIONS

5.1 SITE GEOLOGY

The project site lies within the Coast Ranges Geomorphic Province of California[2], characterized by a series of northwest trending mountain ranges and valleys sub-parallel to the San Andres Fault. The Coast Ranges is composed of thick Mesozoic and Cenozoic sedimentary strata. The northern Coast Ranges is dominated by irregular, knobby, landslide-prone material of the Franciscan Complex. The project site is located within Anderson Valley, an alluvial valley situated within the Coast Ranges.

Published geologic mapping[3] shows Anderson Valley underlain by Quaternary, non-marine, terrace deposits, primarily consisting of sand and gravel with minor amounts of silt and clay. This formation can reach thicknesses of up to 150 feet in the Boonville area based on the California Groundwater Bulletin 118, Anderson Valley Groundwater Basin (2004)[4]. The USDA-NRCS Web Soil Survey[5] shows the site as Boontling loam and Pinole loam in the upper 5 feet of the ground surface with gravel commonly below a depth of 3 feet.

The surrounding areas are mapped as undivided marine, Jurassic-Cretaceous age sedimentary rock with minor inter-bedded shale, sandstone, and conglomerate.

Within the immediate stream channel at the bridge, recent alluvium materials are evident and are observed to generally consist of a mixture of clay, silt, sand, and gravel. Gravel and cobbles in a silty sand matrix were evident in steep slopes eroded away by the creek upstream and downstream of the bridge

A regional geologic map is shown on Figure 3 in Appendix I.

5.2 SURFACE CONDITIONS

Lambert Lane is a two-lane, asphalt paved rural road aligned northeast to southwest. The existing bridge (10C-0146) was built in 1954 and is a single-span concrete slab bridge about 32 feet long and 26 feet wide with timber railings. Tall concrete wall abutments founded on spread footings support the bridge. The bases of the footings are approximately 21.2 feet below the deck (approximate elevation of 367 feet) based on the As-Built plan. Channel scour has exposed the footings at both abutments and the east abutment has been undermined about 3 feet below the footing, exposing coarse cobbles in a silty sand matrix. The bridge is considered scour critical due to footing exposure and undermining.

[2] California Geologic Survey (2002), *California Geomorphic Province*, Note 36.
[3] Jennings, C.W., and Strand, R.G. (1960), Geologic Map of California: Ukiah sheet, 1:250,000, California Division of Mines and Geology
[4] California Department of Water Resources (2004), Anderson Valley Groundwater Basin, California Groundwater Bulletin 118
[5] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture, Web Soil Survey, http://websoilsurvey.nrcs.usda.gov/

Robinson Creek immediately upstream of the bridge makes a sharp, 90-degree curve to the northeast, then turns 90-degrees again to pass through the bridge opening. This channel geometry severely impacts the south approach. The south approach slope is protected by a concrete retaining wall that extends about 50 feet upstream from the wing-wall. About half of the wall (adjacent to the bridge) has failed and fallen into the creek; the remaining wall has cracked and deflected. Large RSP has been placed in the failed wall area and behind the remaining wall section to protect the southern approach. The channel thalweg is at approximate elevation 368 feet, about 24 feet below the existing bridge deck.

There are residential houses near the bridge with the nearest structure being approximately 130 feet west of the bridge. Overhead utility lines are present at the site running parallel on the east side of the road. No underground utilities were identified at the site after calling in an Underground Service Alert (USA) before subsurface exploration commenced. Any underground utilities if present, are unknown.

5.3 SUBSURFACE CONDITIONS

Based on the exploratory boring data, subsurface materials underlying the bridge site are considered consistent with non-marine terrace deposits shown on the published mapping. We divide materials encountered in our borings into two general soil units, as shown in Table 2. Refer to the LOTB in Appendix II for more specific soil descriptions, boring details, and elevations.

Table 2: Subsurface Materials

Unit	Elevation (ft)	Depth (ft)	Generalized Soil/Rock Layer Description
1	388.0 to 356.0	0 to 32.0	Dense to very dense, brown silty, clayey sand, stiff to hard sandy lean clay, and sandy silty clay. Layers of lean clay, silt with gravel and cobbles within R-18-002 and R-18-003. Colors include brown, dark/yellow brown, tan, and variegated brown, red yellow, and light gray.
2	356.0 to 277.0	32.0 to 111.0	Dense to very dense silty sand with gravel; medium dense to very dense silty sand. Colors include brown, tan, and variegated tan/light gray/red yellow. The materials in this unit were drilled with 94-mm mud rotary drill bit to the maximum depth explored in the borings. In all borings below elev. 340 ft in R-18-001 through R-18-003, the sampler penetration predominately did not achieve the full 18-inches of penetration (typically ranging from 2 to 17-inches, average 11-inches) to the full depth of each boring.

6 GROUNDWATER

The groundwater levels encountered/recorded in the geotechnical test borings completed for this project are shown in Table 3. Groundwater was not encountered within the augered portion in boring R-18-001 to about 28 ft depth (elev. 360± ft). Groundwater levels can fluctuate due to changes in precipitation, irrigation, and other factors. Groundwater for geotechincal analysis is modeled at elev. 368.0 ft.

Table 3: Groundwater Data

Support Location	Boring ID	Date	Groundwater Depth/Elevation (feet)
Abutment 1	R-18-002	4/18/2018	20.5/368.0
Abutment 1 (Retaining Wall)	R-18-003	4/19/2018	29.0/360.5
Abutment 2	R-18-001	4/18/2018	Not Measured*

* Groundwater not measured due to presence of residual drill fluid the test boring.

7 AS-BUILT FOUNDATION DATA

The As-built plans (not dated) for the existing bridge show the abutment walls supported on concrete spread footings (bearing capacity not shown). The footing cross-section is shown to be 6’-4” wide and 1’-8” thick, running the width of the bridge at 26’-2”. The plan did not contain information regarding the elevation of the road or creek bed or the embedment depth of the footing with respect to the creek bed. No Log of Test Borings (LOTB) was available for the existing bridge.

8 SCOUR DATA

Scour data for this project was provided by Wreco in their Bridge Design Hydraulic Study Report dated August 5, 2021. The scour data for each abutment is summarized in Table 4.

Table 4: Scour Data

Support	Long-Term (Degradation/Aggradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Abutment 1	372.0	4.6
Abutment 2	373.2	3.4

9 CORROSION EVALUATION

The results of corrosivity tests on soil samples obtained from the borings completed for this project are summarized in Table 5.

Table 5: Soil Corrosivity Test Results

Boring / Sample No.	Depth (ft)	Elevation (ft)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	Corrosive
A-18-002 / 6B	25.5	363.5	7.66	4,290	1.5	3.6	No
A-18-003 / 4B	10.5	376.5	7.46	2,950	6.1	12.2	No
A-18-003 / 7B	20.5	366.5	7.10	4,560	1.3	3.7	No

For structural elements, Caltrans[6] defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1,500 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 1,500 ohm-cm.

Based on the corrosivity test results summarized in Table 3 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' Corrosion Guidelines provides information regarding corrosion mitigation measures for structural elements and lists additional Caltrans guideline documents regarding corrosion mitigation if deemed appropriate by the designer. The designer should also consult with a corrosion engineer if the test result values are considered significant.

10 SEISMIC INFORMATION

10.1 SHEAR WAVE VELOCITY

For this project, a correlated shear wave velocity (V_{S30}) in the upper 100 ft of the soil profile equal to 280 meters per second (about 917 ft/sec) is considered appropriately conservative for use in new bridge design. This value corresponds to a “stiff soil” with 180 m/s < Vs < 360 m/s for the upper 100 ft of the soil profile. The V_{S30} value was determined for this site based on the subsurface data obtained from the test borings and correlations with SPT blow count N-values corrected for hammer efficiency using the equations outlined by Caltrans[7]. For our evaluation, we used latitude 39.0084°N and longitude 123.3683°W for the site coordinates.

The correlated V_{S30} values estimated from the test boring logs are shown in Table 6.

Table 6: Correlated Shear Wave Velocity

Support Location	Boring Designation	Top of Boring Elevation (ft)	Bottom of Boring Elevation (ft)	Total Boring Depth (ft)	Correlated Shear Wave Velocity in Upper 100 ft	
					Vs_{30} (m/sec)	Vs_{30} (ft/sec)
Abutment 1	R-18-002	388.5	277.5	111.0	290	952
Abutment 1 (Retaining Wall)	R-18-003	389.5	328.5	61.0	305	1001
Abutment 2	R-18-001	388.3	297.3	91.0	282	924

[6] Caltrans, Corrosion Guidelines Version 3.2, May 2021.
[7] California Department of Transportation, *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations*, Appendix A, November 2012.

10.2 SOIL CLASSIFICATION

For seismic design, Caltrans classifies soil as either Class S1 or Class S2. The Class S1 soil classification represents competent soil. The Class S2 soil classification represents non-competent soils, including marginal soil, poor soil and soil susceptible to lateral spreading.

According to Caltrans' SDC V2.0, Class S1 soil must meet all of the following criteria:

- Standard Penetration Test, $(N_1)_{60} \geq 30$ (Granular Soils)
- Undrained Shear Strength, $s_u > 2{,}000$ psf (Cohesive Soils)
- Shear Wave Velocity, $v_s > 886$ ft/sec
- Not susceptible to liquefaction, lateral spreading, or scour

Soil that does not satisfy the requirements listed above is to be classified as Class S2 soil.

Based on the boring data and criteria listed above, site soils are classified as Class S2 (non-competent) due to the presence of a few granular soil layers with $(N_1)_{60} < 30$. The simplified design method as specified in Section 6.2.3.2 of Caltrans' SDC V2.0 is not allowed for piles founded in Class S2 soil and lateral analysis as specified in Section 6.2.4.2 of Caltrans' SDC V2.0 is required.

10.3 GROUND MOTION HAZARD

The Caltrans ARS Online (V3.0.2)[8] web-based tool was used to calculate the probabilistic acceleration response spectra for the site based on criteria outlined in Appendix B of Caltrans SDC.

For the purposes of this report, we assume the new bridge is categorized as Ordinary. For Ordinary bridges, the design spectrum is based on the Safety Evaluation Earthquake (SEE) spectrum only. A probabilistic evaluation approach is used to determine the SEE design spectrum taken as the spectrum based on the 2014 USGS Seismic Hazard Map for the 5% in 50 years probability of exceedance (or 975-year return period).

Caltrans structure design practice 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 adjustment factor is applied for locations with a site to rupture plane distance (Rrup) of 25 km (15.6 miles) or less to the causative fault and is based on the deaggregated mean distance for spectral acceleration at a period of 1.0 second. The basin factor and near-fault adjustment factor do not apply to this site.

The mean magnitude value reported by Caltrans ARS Online is not used in the ground motion calculation. It is included to support simplified liquefaction analysis and is obtained from a hazard deaggregation performed at the Peak Ground Acceleration (PGA).

10.3.1 SEISMIC DESIGN DATA AND CALTRANS DESIGN SPECTRUM

Based on the above information, we recommend structure design for an ordinary bridge using the SEE Design Spectrum in accordance with following Caltrans SDC parameters:

[8] https://arsonline.dot.ca.gov/, accessed 04/01/2020.

- Shear Wave Velocity, V_{S30}: 917 feet/second (280 meters/second);
- Peak Ground Acceleration (PGA): 0.52g;
- Design Magnitude (M) at PGA: 7.2; and
- Design Mean Site-to-Source Distance at 1.0 Second: 20.9 mi (33.7 km).

The Design Ground Motion Data Sheet presenting the SEE Design ARS data, curve, and other relevant information is attached as Appendix IV.

10.4 OTHER SEISMIC HAZARDS

10.4.1 SURFACE FAULT RUPTURE

The site does not lie within an Alquist–Priolo Earthquake Fault Zone and no known active faults are mapped by the California Geologic Survey[9] (CGS) within or through the project area. The CGS considers a fault to be active if it has shown evidence of ground displacement during the Holocene period, defined as the last 11,000 years. According to the CGS, the closest active faults are the San Andreas (North Coast) 2011 CFM (~14.8± miles southwest of the site) and the Maacama Fault Zone (North Section) (~13.4± miles northeast of the site). Per Caltrans' Memo to Designer 20-15, the structure is not considered susceptible to surface fault rupture hazards. Nearby faults are shown on Figure 4 in Appendix I.

10.4.2 LIQUEFACTION POTENTIAL

Soil liquefaction can occur when saturated, relatively loose sand and specific soft, fine-grained saturated soils (typically within the upper 50 feet) are subject to ground shaking strong enough to create soil particle separation that results from increased pore pressure. This separation and subsequent pore pressure dissipation can lead to decreased soil shear strength and settlement. Liquefaction is known to occur in soils ranging from low plasticity silts to gravels. However, soils most susceptible to liquefaction are clean sands to silty sands and non-plastic silts. Granular soils with SPT blow count $(N_1)_{60} \geq 30$, rock and most clay soil are not liquefiable.

Based on the dense/very stiff soil layers encountered below groundwater in the borings completed for this study, the potential for liquefaction is negligible at this site and is not a geotechnical consideration for foundation design.

10.4.3 SEISMIC SETTLEMENT

During a seismic event, ground shaking can cause densification of dry loose to medium dense cohesionless soils above the water table that can result in settlement of the ground surface. Based on the consistency of the soils encountered above the water table in the borings completed for this study, the potential for seismically-induced ground settlement is not a geotechnical consideration for project design.

10.4.4 LATERAL SPREADING

Lateral spreading, characterized by incremental flow-failure within liquefiable soil on sloping ground or a free face, is capable of producing horizontal ground displacement during a seismic event. Youd et al. (2002)[10] indicate that potentially liquefiable soil layers with SPT $(N_1)_{60}$ values

9 http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.
10 American Society of Civil Engineers (ASCE) Journal of Geotechnical & Geoenvironmental Engineering, Dec 2002.

greater than 15 are too dense and dilative for lateral spread to occur. Based on the predominantly dense granular soil layers (i.e., $(N_1)_{60} \geq 15$) encountered in the borings completed for this study, the potential for liquefaction does not exist at this site. Therefore, the potential for lateral spreading to occur at this site does not exist and is not a geotechnical design consideration.

10.4.5 SEISMIC SLOPE STABILITY

The potential for seismic instability of the existing creek banks is considered to be low and likely limited to some potential for minor bank distortion. The potential for seismically induced instability on engineered fill slopes, constructed at 1.5:1 with rock slope protection (RSP) per Caltrans Standard Section 72-2 and typical gradients of 2:1 with no RSP or flatter, is considered low.

11 GEOTECHNICAL RECOMMENDATIONS

The site is considered adequately stable with support available for the new bridge by means of deep foundations penetrating into the underlying dense to very dense soils encountered in the borings completed for this study. We understand that 30-inch diameter CIDH piles for the bridge replacement have been selected for use at this site due to environmental constraints to avoid excessive noise/vibration issues associated with pile driving. Specific recommendations for 30-inch diameter CIDH piles are provided below.

Based on the geotechnical data developed for this project, CIDH piles can provide adequate axial capacity and minimize construction noise and vibration. The presence of groundwater is expected at the site within CIDH pile foundation depths during construction. Therefore, we recommend that the CIDH piles be installed by the "wet" method, including slurry drilling and concrete placed under slurry using tremie pipe. The "wet" method requires placement of inspection tubes to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) of the CIDH pile.

The site is also considered adequately stable with shallow foundation support available for the proposed Caltrans Type 1A (Case 1) retaining wall behind Abutment 1. We understand that the Caltrans Standard Retaining Wall Type 1A (Case 1) will have a maximum height of 6 ft and bottom of footing at elev. 382.83 ft. We recommend the use of spread footing foundations bearing on an 18-inch thick prism of engineered fill founded on intact soils for the planned retaining wall.

The site is located within a seismically active region and moderate to high ground shaking can be expected at the site during the design life of the bridge. However, no potentially liquefiable soils or secondary seismic effects (i.e., seismic settlement, seismic slope instability) are identified from the data generated for this study. The risk of fault rupture hazard is not a design consideration for this project. No other over-riding geologic hazards (e.g., soft soils, subsidence, landslides, etc.) were identified by published geologic mapping or site reconnaissance performed for this study/project.

11.1 DEEP FOUNDATIONS (BRIDGE ABUTMENTS)

11.1.1 PILE FOUNDATION DATA AND LOADING

Foundation data and loading for the proposed file foundations provided by QEI are shown in Table 7 and 8.

Table 7: Foundation Design Data

Support No.	Design Method	Pile Type	Finished Grade Elevation[1] (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement – Service Load (in)[2]	Number of Piles per Support
					B	L		
Abut 1	LRFD	30-inch CIDH	390.00	374.5	11	38.67	2	9
Abut 2	LRFD	30-inch CIDH	390.66	374.5	11	38.67	2	9

[1] Indicates the finished grade at the front face of the supports.
[2] Based on Caltrans' current practice, the total permissible settlement 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 type abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Table 8: Foundation Factored Design Loads

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
			Compression		Tension		Compression		Tension	
	Total Load Per Support	Permanent Loads Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	1350	1130	1900	280	NA	NA	NA	NA	NA	NA
Abut 2	1350	1130	1900	280	NA	NA	NA	NA	NA	NA

11.1.2 PILE FOUNDATION DESIGN RECOMMENDATIONS

The foundation design recommendations for 30-inch CIDH piles at the abutments are summarized in Table 9.

Table 9: Foundation Design Recommendations

Support Location	Pile Type	Cut-off Elev. (ft)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (inches)	Nominal Resistance[4] (kips)				Design Tip Elev. (ft)	Specified Tip Elevation (ft)
						Strength/Const.		Extreme			
			Total	Perm.		Comp. φ = 0.7	Tens. φ = 0.7	Comp. φ = 1.0	Tens. φ = 1.0		
Abut 1	30" CIDH	374.5	1350	1130	2	400	NA	NA	NA	314 (a-I) 338 (b)	314
Abut 2	30" CIDH	374.5	1350	1130	2	400	NA	NA	NA	314 (a-I) 338 (b)	314

Notes:
1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit) and (b) Lateral Load.
2) The Specified Tip Elevation should not be raised above the design tip elevation.
3) The Design Tip Elevation for the Lateral Load was determined by QEI.
4) Column heading modified from Required Factored Nominal Resistance to Nominal Resistance.
5) The piles will be embedded adequately into dense/hard soils, and the piles will not be subjected to downdrag loads, a detailed assessment of the pile group settlement is not considered warranted.

No seismic downdrag is expected and is therefore not a geotechnical design consideration.

Refer to Appendix V for our foundation design calculations that include geotechnical design parameters, assumptions, methodology, and summarizes the results of our pile compression resistance, tension resistance, settlement, negative skin friction, and group reduction analyses.

11.1.3 PILE DATA TABLE

The recommended Pile Data Table is presented as Table 10.

Table 10: Pile Data Table

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevations (ft)
		Compression	Tension		
Abut 1	30" CIDH	400	NA	314 (a) 338 (b)	314
Abut 2	30" CIDH	400	NA	314 (a) 338 (b)	314

Notes:
1) Design tip elevations are controlled by: (a) Compression and (b) Lateral Load.
2) The Specified Tip Elevation should not be raised above the design tip elevation.
3) The Design Tip Elevation for the Lateral Load was determined by QEI.
4) The piles will be embedded adequately into dense/hard soils, and the piles will not be subjected to downdrag loads, a detailed assessment of the pile group settlement is not considered warranted.

11.1.4 LATERAL RESISTANCE

The parameters for LPile are provided in Appendix V for QEI's use in lateral pile capacity analysis.

For multiple row shading apply appropriate P-Multipliers (Pm) shown in Table 11 consistent with Section 10.7.2.4 of the April 2019 California Amendments to AASHTO LRFD Bridge Design Specifications (8th Edition). Interpolate the P-Multiplier value for intermediate pile spacing. For loading direction and spacing refer to Figure 10.7.2.4-1 of AASHTO LRFD Bridge Design Specifications (8th Edition).

Table 11: Pile P-Multipliers, P_m for Multiple Row Shading

Pile CTC spacing (in the Direction of Loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
2.0B	0.60	0.35	0.25
3.0B	0.75	0.55	0.40
5.0B	1.0	0.85	0.70
7.0B	1.0	1.0	0.90

CTC = Center-To-Center Spacing / B = Pile Diameter / Use a P-Multiplier = 1.0 for Pile CTC Spacing ≥ 8B

Where the loading direction is perpendicular to a single row of piles use a P-Multiplier of 0.80, 0.90, and 1.0 for pile spacing of 2.5B, 3B and ≥4B, respectively.

Lateral pile analysis should also incorporate change in foundation due to limit state for scour in accordance with the latest California Amendments to Section 3.7.5 of AASHTO LRFD Bridge Design Specifications (8th Edition).

11.2 SHALLOW FOUNDATIONS (RETAINING WALL)

11.2.1 ENGINEERED FILL PRISM

We recommend that spread footing foundation support for the Caltrans Standard Type 1A (Case 1) retaining wall be established within a prism of engineered fill to provide suitably firm, uniform support. All soil should be excavated to at least 18-inches below base of footing. Horizontal limits of excavation should be from heel line to 2 feet in front of the toe of retaining wall footing. If right-of-way is less than 2 ft, then it is acceptable to limit the horizontal limit of excavation to the right-of-way line. The overexcavation limits should be shown on the plans.

The excavated area should be filled to top of footing grade with Structure Backfill (per Caltrans "Standard Specifications") compacted to at least 95% relative compaction (per CTM 216). Use of Caltrans Class 2 Aggregate Base may be used in lieu of Structure Backfill.

11.2.2 SPREAD FOOTING FOUNDATION DATA AND LOADING

The retaining wall spread footing foundation data and loading from the Caltrans 2018 Standard Plan (SP) sheet B3-3A is summarized in Tables 12.

Table 12: Design Data for Caltrans Type 1A (Case 1) Retaining Wall

ERS Station Interval (ft)	Design Height (ft)	Bottom of Footing Elev. (ft)	Footing Width (ft)	Minimum Footing Embedment Depth (ft)	Limit State	Effective Footing Width (B') (ft)	Gross Uniform Bearing Stress (ksf)	Net Bearing Stress (ksf)	Total Permissible Settlement (in)
10+00 to 10+28 "RW1" Line	6.0	382.83	7.0	1.5	Service	6.7	NA	1.0	2.0
					Strength	5.2	1.7	NA	NA
					Extreme I	4.8	1.4	NA	NA
					Extreme II	2.7	2.5	NA	NA

11.2.3 SPREAD FOOTING FOUNDATION RECOMMENDATIONS

The spread footing foundation design recommendations for the planned Caltrans Type 1A (Case 1) retaining wall are summarized in Table 13. Refer to Appendix V for our foundation design calculations that include geotechnical design parameters, assumptions, methodology, and summarizes the results of our spread footing bearing resistance and settlement.

Table 13: Spread Footing Design Recommendations

ERS Station Interval (ft)	Design Height (ft)	Bottom of Footing Elev. (ft)	Footing Width (ft)	Limit State	Effective Footing Width (B') (ft)	Geotechnical Resistance Factor[1]	Factored Bearing Resistance (ksf)	Calculated Settlement at Net Bearing Pressure (inch)
10+00 to 10+28 "RW1" Line	6.0	382.83	7.0	Service	6.7	1.00	NA	0.25
				Strength	5.2	0.55	3.5	NA
				Extreme I	4.8	0.80	4.7	NA
				Extreme II	2.7	0.80	2.6	NA

1. Table 11.5.7-1 Resistance Factors for Permanent Retaining Walls, California Amendments to AASHTO LRFD Bridge Design Specifications, 8th Edition.

11.2.4 SLIDING RESISTANCE

To evaluate shallow foundation sliding resistance at the strength limit state consistent with Section 10.6.3.4 of the AASHTO LRFD BDS and current Caltrans amendments, we recommend a friction angle (ϕ_f) equal to 34° corresponding to a friction factor ($\tan \delta = \tan 2/3\phi_f$) of 0.42 for cast in-place concrete foundations bearing on engineered fill prism. Use a resistance factor (ϕ_τ) of 1.0[11] for shear resistance between the bottom of the cast-in-place concrete retaining wall footing and bearing material. No passive pressure is allowed.

For retaining wall design a resistance factor equal to 1.0 is also used to calculate sliding resistance for the service and extreme limit states.

[11] Table 11.5.7-1 Resistance Factors for Permanent Retaining Walls, Section 11: Walls, Abutments, and Piers, California Amendments to AASHTO LRFD Bridge Design Specifications, 8th Edition.

11.2.5 SPREAD FOOTING DATA TABLE

Table 14 presents the Spread Footing Data Table for the planned Caltrans Type 1A (Case 1) retaining wall supported in/on an engineered fill prism as recommended above.

Table 14: Spread Footing Data Table for Caltrans Type 1A (Case 1) Retaining Wall

ERS Station Interval (ft)	Design Height (ft)	Service Permissible Net Contract Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (ϕ_b = 0.55) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (ϕ_b = 0.80) (ksf)
10+00 to 10+28 "RW1" Line	6.0	10.0	3.5	2.6

11.3 LATERAL EARTH PRESSURES

The material placed behind each abutment/wingwall and the retaining wall is expected/recommended to meet Structure Backfill requirements consistent with Caltrans 2018 Standard Specifications. The equivalent fluid weights (EFWs) shown in Table 15 are recommended to design the abutment and wing walls (assuming fully drained and level backfill conditions).

Table 15: Recommended Equivalent Fluid Weights (EFW)

Condition	Static		Incremental Seismic	
	Coefficient k (dim.)	EFW (pcf)	Coefficient Δk (dim.)	Δ EFW_{EQ} (pcf)
Active	0.28	34	0.11	13
At-Rest	0.44	53	NA	36

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[12] and Specifications.

11.3.1 STATIC LATERAL EARTH PRESSURE

Caltrans allows use of static active earth pressure for embankment behind seat-type abutments. The static at-rest condition is recommended for walls that are restrained at the top and cannot

[12] Caltrans, "Standard Plans," 2018.

deflect or move (e.g., behind diaphragm-type abutments). A triangular pressure distribution should be used and applied to the controlling static resultant earth pressure at a distance of H/3 from the base of the wall.

11.3.2 SEISMIC LATERAL EARTH PRESSURE

The active seismic coefficient was calculated using the Mononobe-Okabe (M-O) equation presented in Appendix A11 of AASHTO BDS and horizontal seismic acceleration coefficient (k_h) value of 0.17 (i.e., approximately one-third of the site PGA consistent with Caltrans Geotechnical Manual, January 2021). The incremental at-rest seismic pressure (ΔEFW_{EQ}) was estimated using guidance from the University of California at Berkeley for Caltrans[13] and the site PGA of 0.52g. A triangular pressure distribution should be used and the magnitude of the resultant should be applied at 0.5H from the base of the wall.

11.3.3 SURCHARGE LOADS

For surcharge loads, apply an additional uniform lateral load behind the wall that is the greater of the values shown in Table 16 for the applicable earth pressure condition.

Table 16: Surcharge Loads

Active Condition	At-Rest Condition
0.28-times the design surcharge pressure[1]	0.44-times the design surcharge pressure[1]

[1]The minimum surcharge pressure should be 240 psf, which represents an equivalent height of soil equal to 2 ft where the unit weight of soil is 120 pcf.

11.4 APPROACH FILLS

Site grading and general earthwork for the approach roadway should be performed in accordance with Section 17 and Section 19 of Caltrans 2018 Standard Specifications.

The planned approach fills within 150 feet of the bridge abutments are limited to 5 foot thickness or less and no special considerations for soft or otherwise unsuitable soil is indicated from the boring data generated for this study.

11.5 STRUCTURAL SECTION AND ROADWAY SUBGRADE

We completed an R-value test (CTM 301) on a bulk sample of anticipated subgrade soils. The test result indicates an R-value of 66 by Stabilometer. A design R-value of 50 corresponding to the minimum R-value for Class-2 Aggregate Subbase per Caltrans "Standard Specifications" is recommended for new pavement structural section design. Imported fill used at and below subgrade level should be non-expansive and be required to meet or exceed that of the design R-value.

Recommended 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 = 50 are shown in Table 17.

[13] Mikola, R.G. & Sitar, N., Seismic Earth Pressures on Retaining Structures in Cohesionless Soils, March 30, 2013.

Table 17: New Pavement Structural Sections

Section	Material	Traffic Index (TI) R-value = 50						
		4.0	5.0	6.0	7.0	8.0	9.0	10.0
Hot Mix Asphalt (HMA) over Class 2 Aggregate Base (AB)	HMA (feet)	0.15	0.20	0.25	0.30	0.40	0.45	0.50
	AB (feet)	0.20	0.30	0.35	0.45	0.45	0.55	0.65

Asphalt pavement thicknesses shown above are minimum depths and incorporate a 0.2-foot Gravel Equivalent factor of safety in accordance with Caltrans flexible pavement design methods. Other flexible pavement structural sections, typically involving variation 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 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 Caltrans Class 2 Aggregate Base compacted to at least 90% relative compaction per CTM 216. Native granular soils – less debris, organic material and particles over 4-inches greatest dimension – are considered suitable for use as compacted fill.

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.

12 NOTES FOR SPECIFICATIONS

This section is provided to assist the designer develop the geotechnical related Standard Special Provisions (SSPs) for this project element. Before using the information provided in this section, the designer should read and review the report to comprehend the contents and intent of the geotechnical design.

For the project described herein, we recommend the foundation report, log of test borings and legend, and any subsequent addenda be included with project documents during the bidding process for reference purposes.

12.1 GENERAL CONSIDERATIONS

Bridge and retaining wall construction should conform to Caltrans 2018 Standard Specifications unless otherwise stated in the “Special Provisions”. We include the following Caltrans Standard Specification sections as a minimum (non-comprehensive) to be included into the project plans and specifications.

- Section 17 -2 Clearing and Grubbing
- Section 19 Earthwork
- Section 47 Earth Retaining Structures
- Section 19-3.03B(1) Structure Excavation: General
- Section 19-3.03E(1) Structure Backfill: General
- Section 49-1 Piling General
- Section 49-3 Cast-In-Place Concrete Piling
- Section 49-3.02 Cast-In-Drilled-Hole Concrete Piling
- Section 90-1.02G(6) Mixing and Transporting Concrete: Quantity of Water/Slump

The project specifications should explicitly exclude vibration and impact installation methods if noise or vibration are of concern or otherwise not allowed due to environmental constraints, proximity of nearby residences or to protect existing facilities (e.g., underground utilities potentially susceptible to vibration damage).

For the project described herein, we also recommend that the foundation report, log of test borings and legend, and any subsequent addenda be included with project documents during the bidding process for reference purposes.

12.1.1 DEEP FOUNDATIONS (BRIDGE ABUTMENTS)

Due to the anticipated presence of groundwater at this site, we recommend installing CIDH piles by the “wet” method, including slurry drilling and concrete placed under slurry using tremie pipe. We recommend constructing CIDH piles in conformance with Section 49-3 of the 2018 Caltrans Standard Specifications, Revised Standard Specifications, and Standard Special Provisions. The slurry construction method (“wet” method) requires placement of inspection tubes to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) of the CIDH pile (Caltrans Memo to Designers 3-1, June 2014).

For CIDH piles with center-to-center spacing less than 4.0 diameters, the sequence of shaft installation should be specified in the contract documents (Section 10.8.1.2, California Amendments to AASHTO LFRD BDS).

Add to Section 49-3.02C(1):

If the piling center-to-center spacing is less than 4 pile diameters, do not drill holes or drive casing for an adjacent pile until 24 hours have elapsed after concrete placement in the preceding pile and your prequalification test results for the concrete mix design show that the concrete will attain at least 1800 psi compressive strength at the time of drilling or driving.

Drilling equipment must be equipped with instrumentation to measure accurately the actual downward force in pounds. Instrumentation must be visible for reading.

12.2 EXISTING UTILITIES

Overhead utility lines are present at the site running parallel on the east side of the road. No underground utilities were identified at the site after calling in an Underground Service Alert (USA) before subsurface exploration commenced. Any underground utilities if present, are unknown. The presence/absence of any underground utilities should be confirmed prior to construction. Utilities should be protected during construction.

12.3 EXISTING FOUNDATIONS

The existing structure is founded on spread footing foundations as shown on the cited as-built plans. Their removal could result in disturbed soils at/near their locations; however, the new abutments are planned to be behind the current ones and should not affected. As long as they do not conflict with the proposed pile foundations/bridge construction, we recommend removing all concrete footings and existing piles to 3 ft below streambed or finished grade.

13 NOTES FOR CONSTRUCTION

13.1 DEEP FOUNDATIONS (BRIDGE ABUTMENTS)

The contractor should anticipate variable drilling conditions in all CIDH pile excavations due to the presence of medium dense to very dense and very stiff to hard soils with local zones of cobbles. The boring data indicate difficult sampler penetration in earth materials throughout the soil profile. In all borings below about elev. 340, the drive samplers predominately did not achieve the full 18-inches of penetration (ranging from 2 to 17-inches, average 11-inches) to the full depth of each boring. Variable drilling conditions include alternating between soft and hard drilling techniques. The contractor should be prepared to use aggressive drilling equipment (e.g., rock auger bit) to advance the drilled hole excavation.

The calculated *Nominal Axial Resistance* of the CIDH concrete piles is based on skin friction only. End bearing contribution was not used. The zones used to calculate the skin friction of the CIDH concrete piles are shown in Table 18.

Table 18: CIDH Concrete Pile Skin Friction Zones

Support Location	Top of Skin Friction Zone Elevation (ft)	Bottom of Skin Friction Zone Elevation (ft)
Abut 1	369.5	317.5
Abut 2	369.5	319.5

Permanent casing for CIDH pile installation should not be used since the piles would not meet the required nominal bearing resistance due to reduced skin friction. The contractor is responsible for the design and installation of temporary casing (if used), including actual length(s) and diameter(s), to install CIDH piles according to the above specifications without defects or damages to existing utilities/facilities.

Temporary casing (if used), should be noncorrugated steel with smooth surfaces and the casing diameter should be at least 8-inches greater than the CIDH pile to help prevent binding of the drilling tool. Installing temporary casing below the specified pile tip elevation is not permissible.

Driving casing is not permitted for this project/site. If utilized, the temporary casing should be set in a drilled hole and should be removed during placement of concrete. Alternatively, if an oscillator or rotator is used to construct the CIHD piles, the following is recommended:

- The contractor should be prepared for subsurface soil conditions that include layers of medium dense to very dense coarse grained soils and very stiff to hard fine grained soils.

- The contractor should maintain a positive fluid head within the drill rod at all times. The fluid should be mineral or polymer slurry; water is not permitted.
- The contractor should maintain a minimum 10-ft soil plug within the drill rod. The 10-ft plug should be maintained until the drill rod reaches the specified tip elevation. It is recommended that the contractor should not have less than the minimum 10-ft soil plug until the specified tip elevation has been reached. It may be necessary to extend the casing below the bottom of the pile tip to maintain a soil plug to help avoid instability at the base.
- The contractor should provide access to the top of the oscillator/rotator drill rod, as requested by the Engineer, to verify the positive head and minimum soil plug are being maintained.
- It is important to maintain continuous rotation/oscillation and place rebar/concrete expeditiously to avoid lockup. For sites with cohesive soil layers, the contractor should take into account the work window allowed by the plans/specifications to install foundations when proposing vibrator/oscillator method of installation.

The CIDH piles are designed to obtain their geotechnical capacity in side resistance. Nonetheless, the bottom of drilled holes should be cleaned in accordance with Section 49-3.02C(2), “Drilled Holes,” of Caltrans Standard Specifications. Prior to approval, the Engineer should verify the bottom of drilled holes are cleaned before placement of concrete.

Excavation of the CIDH piles, placement of the rebar cage, and concrete pour should be completed in a continuous operation.

Prior to mobilization to the site, the foundation contractor should prepare a Pile Installation Plan in accordance with Section 49-3.02A(3)(b) of the Caltrans Standard Specifications. The work plan should state explicitly any assumptions that the contractor has made regarding earth materials and foundation construction conditions. The work plan should include details of proposed tools/equipment, personnel, materials, methods and order of work. The actual tools and equipment used during CIDH pile excavation/installation should be documented in the construction records. If oscillator/rotator method is used, the contractor’s workplan should include/outline measures to extract a seized casing without compromising the integrity of the CIDH excavation.

13.2 SHALLOW FOUNDATIONS (RETAINING WALL)

Earthwork activities may disturb soils below excavation depths and variation in subgrade support characteristics may be encountered. Prior to placement of fill for the engineered fill prism, a geotechnical engineer or engineering geologist should review the excavation for suitability to receive fill. If loose, disturbed, or otherwise unsuitable materials are present at the exposed subgrade, they should be removed to full depth and replaced with “Structure Backfill”, Class-2 Aggregate Base, or other granular soil as approved by the engineer.

All fill/backfill within the influence of the structure should be placed in 6 to 8-inch loose lifts and compacted to at least 95% compaction (per CTM 216) at over-optimum moisture content. Place structural footing concrete neat (without forming), against trimmed walls and engineered fill at the bottom of the footing excavation. Clean foundation excavations of debris and loose materials prior to placing concrete.

If forming is necessary, backfill excavations outside footing limits with lean concrete or suitable granular backfill (i.e. "Structure Backfill" per Caltrans "Standard Specifications") compacted to at least 95% relative compaction (per CTM 216). Slope the ground surface away from the structures to direct surface runoff away from the foundations.

13.3 EXCAVATION AND SHORING

Based on the subsurface conditions at this site, we expect that excavation to the indicated pile cut-off and retaining wall footing depths can be achieved using typical heavy-duty construction equipment.

Temporary shoring will likely be needed for construction excavations. Use of impact hammers is not expected to be permitted for this project due to environmental considerations. Regardless, driving/vibrating steel sheet piling through very dense soils (including cobbles and boulders) is expected to be difficult with potential for refusal at relatively shallow depth. The need for internal bracing elements (e.g., struts, walers, etc.) would be expected to support temporary sheet pile shoring to help prevent loose materials from entering excavations. A temporary soldier pile wall with CIDH pile elements (similar to proposed pile foundations at the bridge abutments) is considered feasible. Casing may be needed to help control borehole stability and or seepage.

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 personnel, existing structures, utilities and other facilities during construction.

13.4 DEWATERING

Soils below groundwater level are expected to be saturated and capable of transmitting substantial quantities of seepage to open excavations. A Type D structure excavation (per Caltrans) is considered appropriate to show on the plans. Adequate construction dewatering at the bottom of proposed abutment pile footings/caps is expected to be achievable when the creek is dry or during dry season construction (approximately June through October). Winter or spring construction can expect higher water surface level in the creek and may also encounter higher/perched groundwater levels that could require additional controls.

The contractor is responsible for dewatering and/or diking diversion design and construction methods. The contractor should be required to submit excavation, shoring and de-watering plans for review prior to commencing excavations.

14 RISK MANAGEMENT

Our experience, and that of our profession, clearly indicates the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the Geotechnical Engineer of Record to provide additional services during design and construction. For this project, Crawford should be retained as the Geotechnical Engineer of Record to:

- Review and provide 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;

- Observe pile installation and retaining wall excavation during construction in order to verify/confirm anticipated bearing materials, geotechnical resistance, and provide additional or alternate recommendations if necessary; and,
- Update this report if design changes occur, 2 years or more lapse between this report and construction, and/or site conditions have changed.

Should there be significant change in the project or should soil 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.

Crawford 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.

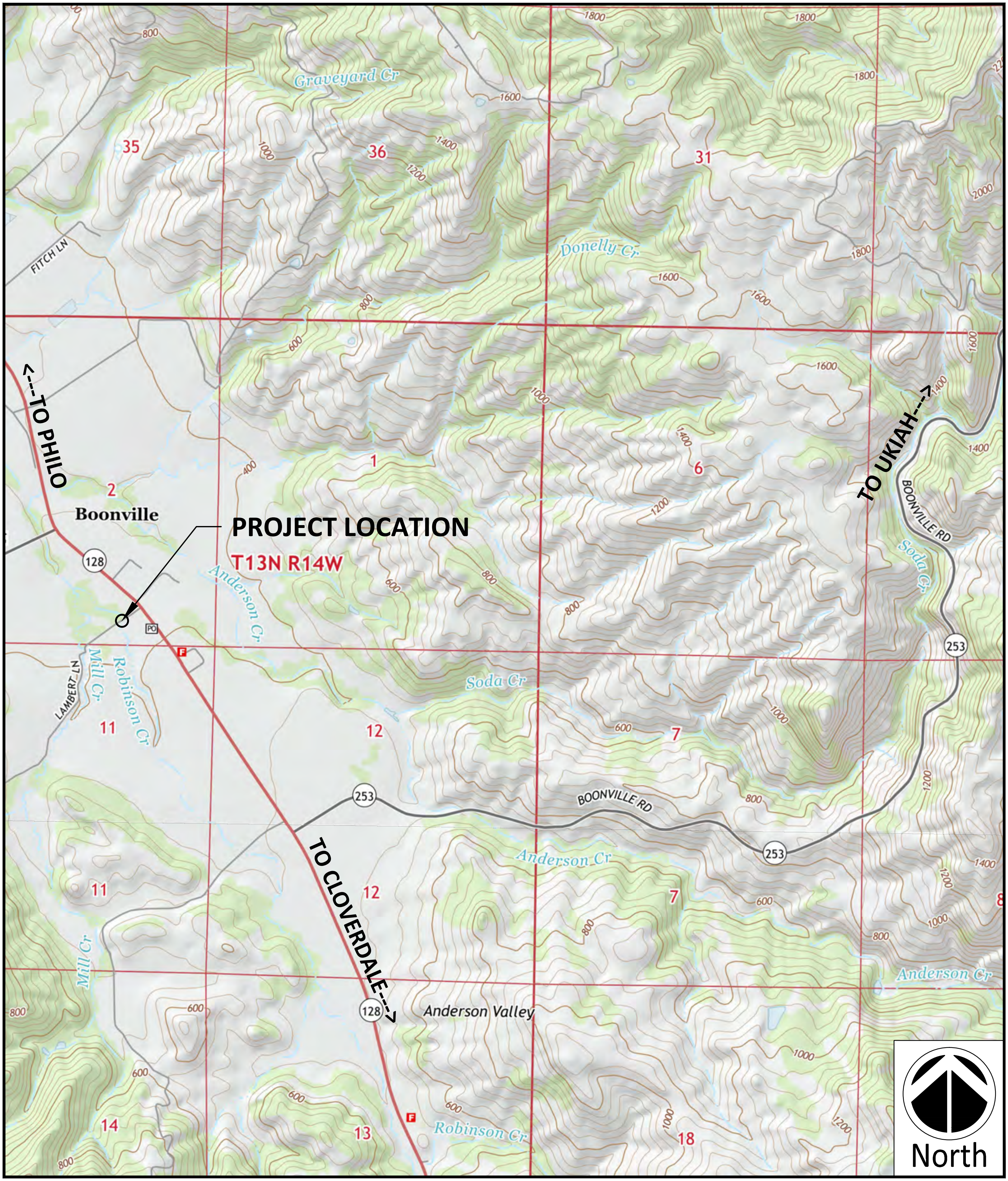
Subsequently defined construction observation procedures and/or agencies are an element of work, which may affect supplementary recommendations.

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 site.

APPENDIX I

Figure 1: Vicinity Map
Figure 2: Exploration Map
Figure 3: Geologic Map
Figure 4: Fault Map



Project Mgr.		
Project Eng.		
Designer		
Checked By		
Drawn By	RRH	05/23/2016
	By	Date

Map Sources

-USGS 7.5' Quadrangle, Boonville, Mendocino County, California, 2015

-USGS 7.5' Quadrangle, Ornbaun Valley, Mendocino County, California, 2015

Crawford & Associates, Inc.
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Sacramento, CA 95831
(916) 455-4225

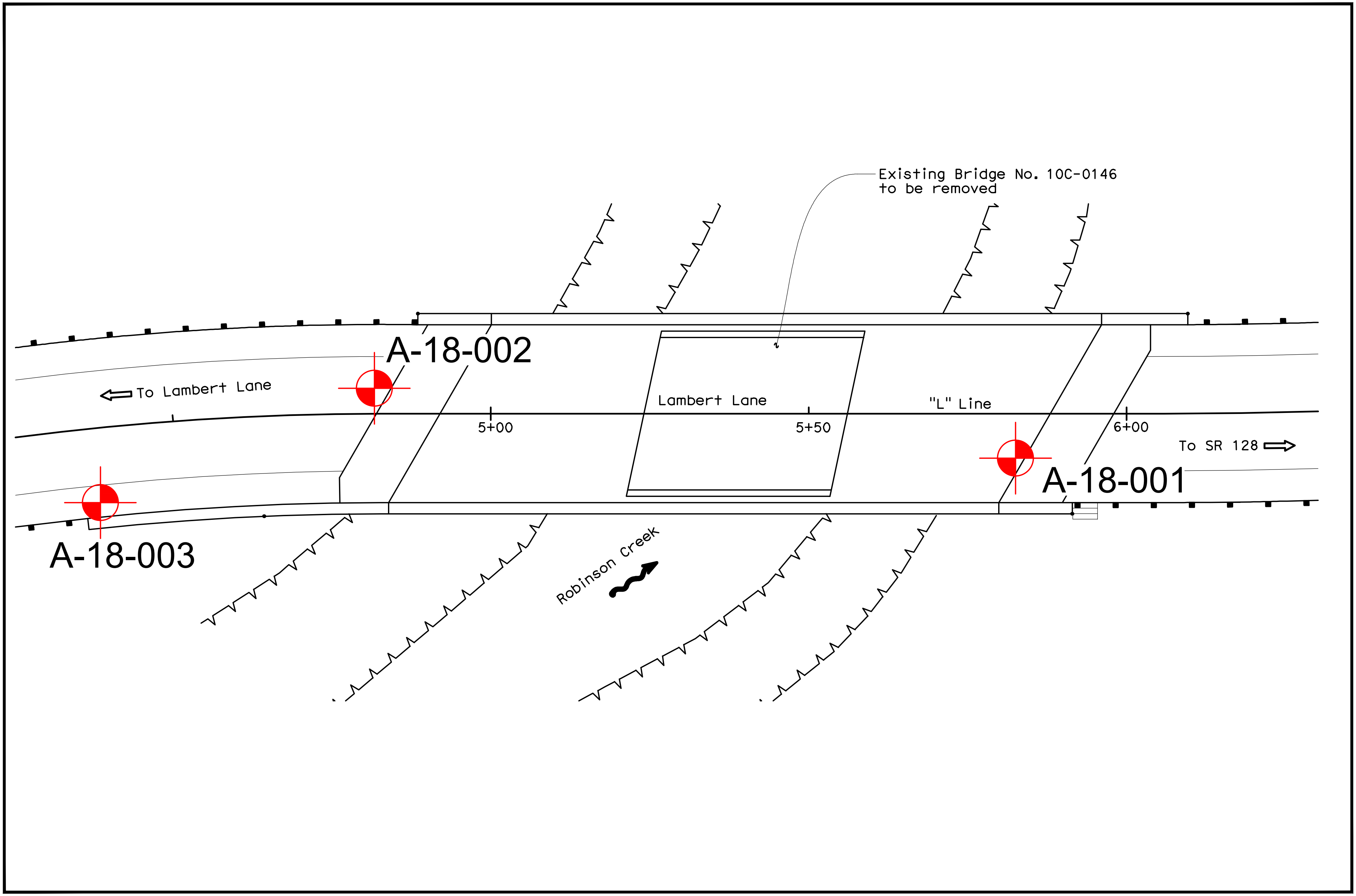
ROBINSON CREEK BRIDGE
REPLACEMENT ON LAMBERT LANE
BRIDGE #10C-146

BOONVILLE, MENDOCINO COUNTY, CA

Figure 1
Vicinity Map

Project No. 16-278.1
Scale 1"=2000'
Date 05/23/2016

Xrefs: 16-278-XC-BORDER-8X11.dwg
Path: S:\AutoCAD Civil 3D\Projects\16-278.1-Robinson Creek Bridge\16-278-ROBINSON CREEK BRIDGE.dwg Plot Date: May 23, 2016 at 11:52am



Source: General Plan plotted on 1/23/2018 from Project Plans for Construction on Lambert Lake; received from QEI.

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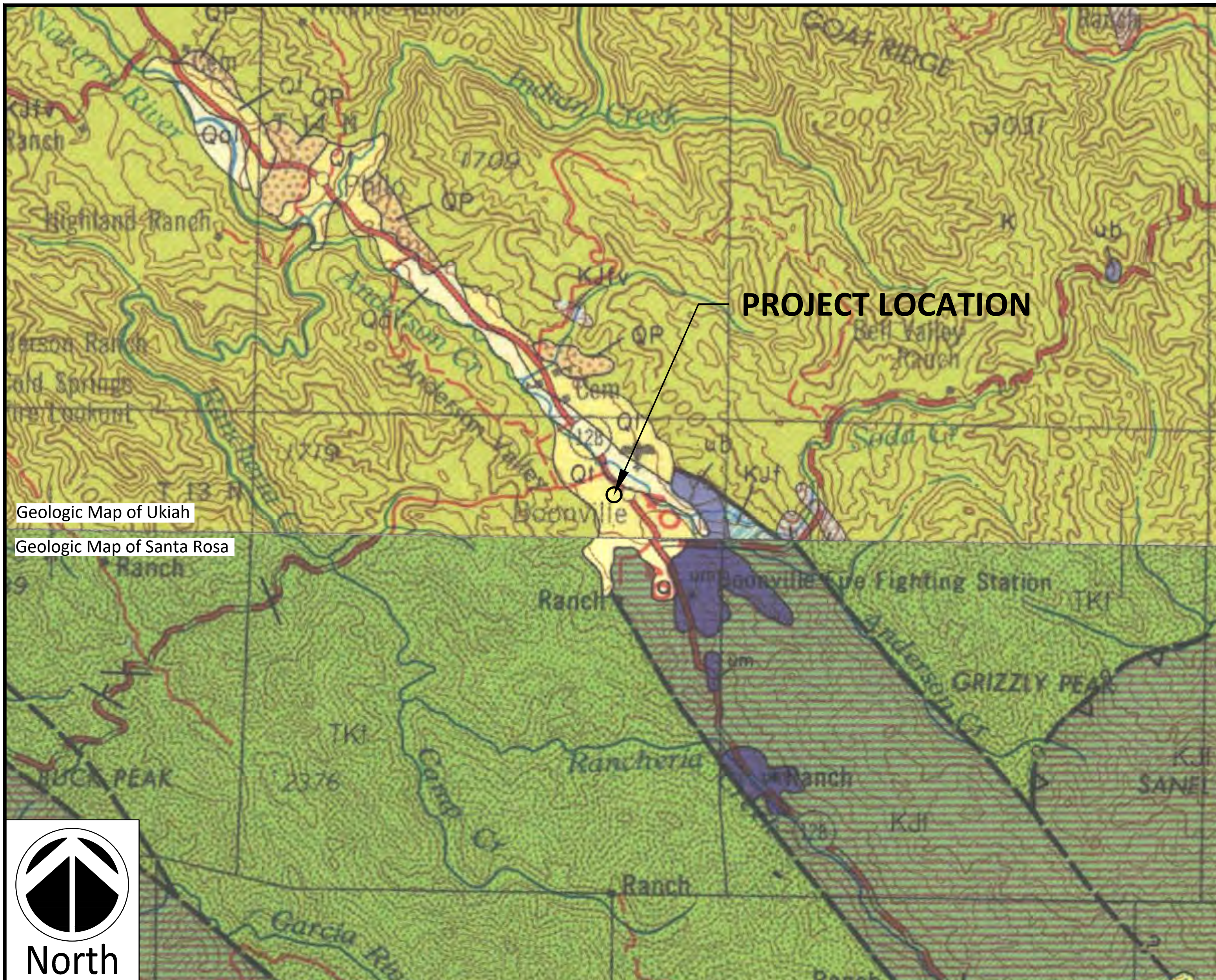
ROBINSON CREEK BRIDGE
REPLACEMENT ON LAMBERT LANE
BRIDGE #10C-146

BOONVILLE, MENDOCINO COUNTY, CA

Figure 2
Boring Exploration Map

Proj. No: 16-278.1
Scale: 1"=20'
Date: 01/21/2022



LEGEND

Geologic Formations (Ukiah Map)

Qal — **Recent Alluvium (Quaternary) -** *Alluvium; Recent breccia, conglomerate and sand of C. A. Anderson; valley fill*

Qt — **Non-marine Terrace Deposit (Quaternary) -** *River and stream terrace deposits*

K — **Undivided Marine Sedimentary Rocks (Cretaceous) -** *Unnamed Cretaceous or Upper Jurassic - sandstone, shale and conglomerate*

Geologic Formations (Santa Rosa Map)

Q — **Alluvium (Quaternary)**

TKf — **Coastal Belt Franciscan (Cretaceous-Lower Tertiary) -** *Marine sandstone, shale and conglomerate*

KJf — **Franciscan Complex* (Jurassic) -** *sandstone, shale, conglomerate, chert, greenstone, and metagraywacke*

um — **Serpentinized Ultramafic Rocks* (Jurassic)**

Horizontal pattern denotes "melange" terrane

Project Mgr.		
Project Eng.		
Designer		
Checked By		
Drawn By	RRH	05/23/2016
	By	Date

Map Sources

1. Jennings, C.W., and Strand, R.G., 1960, Geologic map of California : Ukiah sheet: California Division of Mines and Geology, , scale 1:250,000

2. Wagner, D.L., and Bortugno, E.J., 1982, Geologic map of the Santa Rosa quadrangle, California, 1:250,000: California Division of Mines and Geology, Regional Geologic Map 2A, scale 1:250,000

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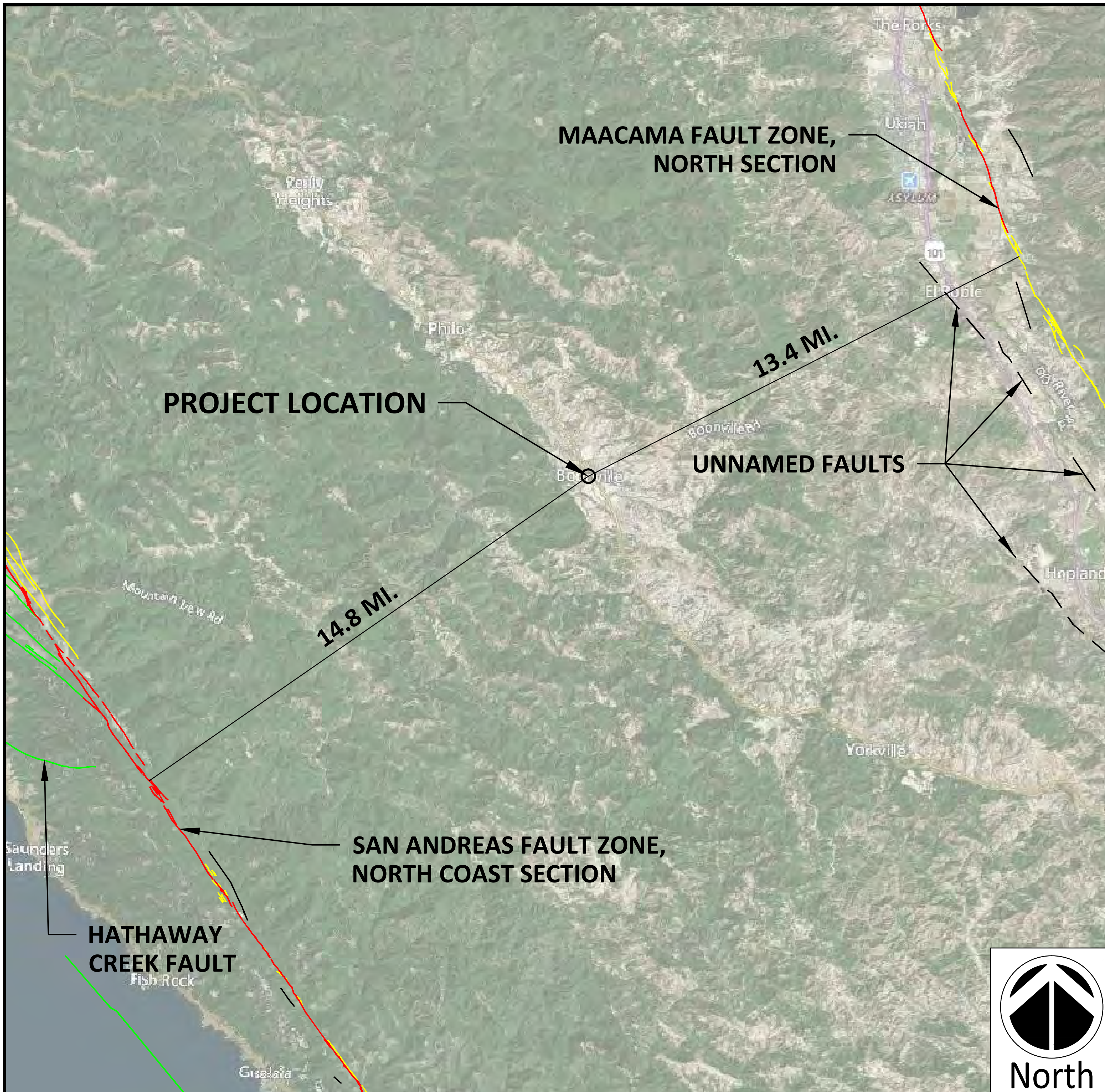
ROBINSON CREEK BRIDGE
REPLACEMENT ON LAMBERT LANE
BRIDGE #10C-146

BOONVILLE, MENDOCINO COUNTY, CA

Figure 3
Geologic Map

Project No. 16-278.1
Scale 1"=10,000'
Date 05/23/2016

Xrefs: 16-278-XC-BORDER-8X11.dwg
Path: S:\AutoCAD Civil 3D\Projects\16-278.1-Robinson Creek Bridge\16-278-ROBINSON CREEK BRIDGE.dwg Plot Date: Jun 20, 2016 at 2:31pm



LEGEND

USGS Faults (Age)	USGS Faults (Age)	Fault Location
<150 years (Historic)	<750,000 years (Mid-Late Quaternary)	Well Constrained
<15,000 years (Holocene)	<1.6 million years (Quaternary)	Moderately Constrained
<130,000 years (Late Quaternary)		Inferred

Project Mgr.		
Project Eng.		
Designer		
Checked By		
Drawn By	RRH	05/23/2016
	By	Date

Map and Data Sources

-Base map via AutoCAD Civil3D Geolocation Map tool

-Quaternary Fault Lines via USGS GIS data

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ROBINSON CREEK BRIDGE
REPLACEMENT ON LAMBERT LANE
BRIDGE #10C-146

BOONVILLE, MENDOCINO COUNTY, CA

Figure 4
Fault Map

Project No. 16-278.1	
Scale	1"=20,000'
Date	05/23/2016

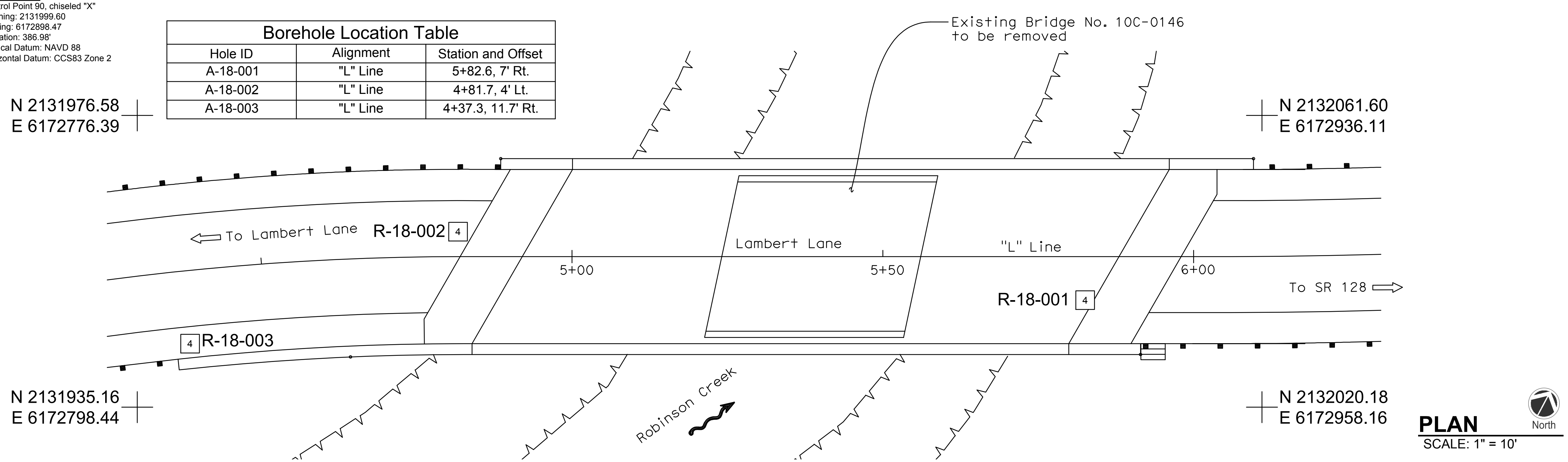
APPENDIX II

Log of Test Borings

BENCH MARK
Control Point 90, chiseled "X"
Northing: 2131999.60
Easting: 6172898.47
Elevation: 386.98'
Vertical Datum: NAVD 88
Horizontal Datum: CCS83 Zone 2

Borehole Location Table

Hole ID	Alignment	Station and Offset
A-18-001	"L" Line	5+82.6, 7' Rt.
A-18-002	"L" Line	4+81.7, 4' Lt.
A-18-003	"L" Line	4+37.3, 11.7' Rt.



PLAN
SCALE: 1" = 10'

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
01	Men	X	X	32	33

REGISTERED CIVIL ENGINEER 1/19/22 DATE

NOT FOR CONSTRUCTION
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER KEIKO K. LEWIS No. C89543 CIVIL STATE OF CALIFORNIA

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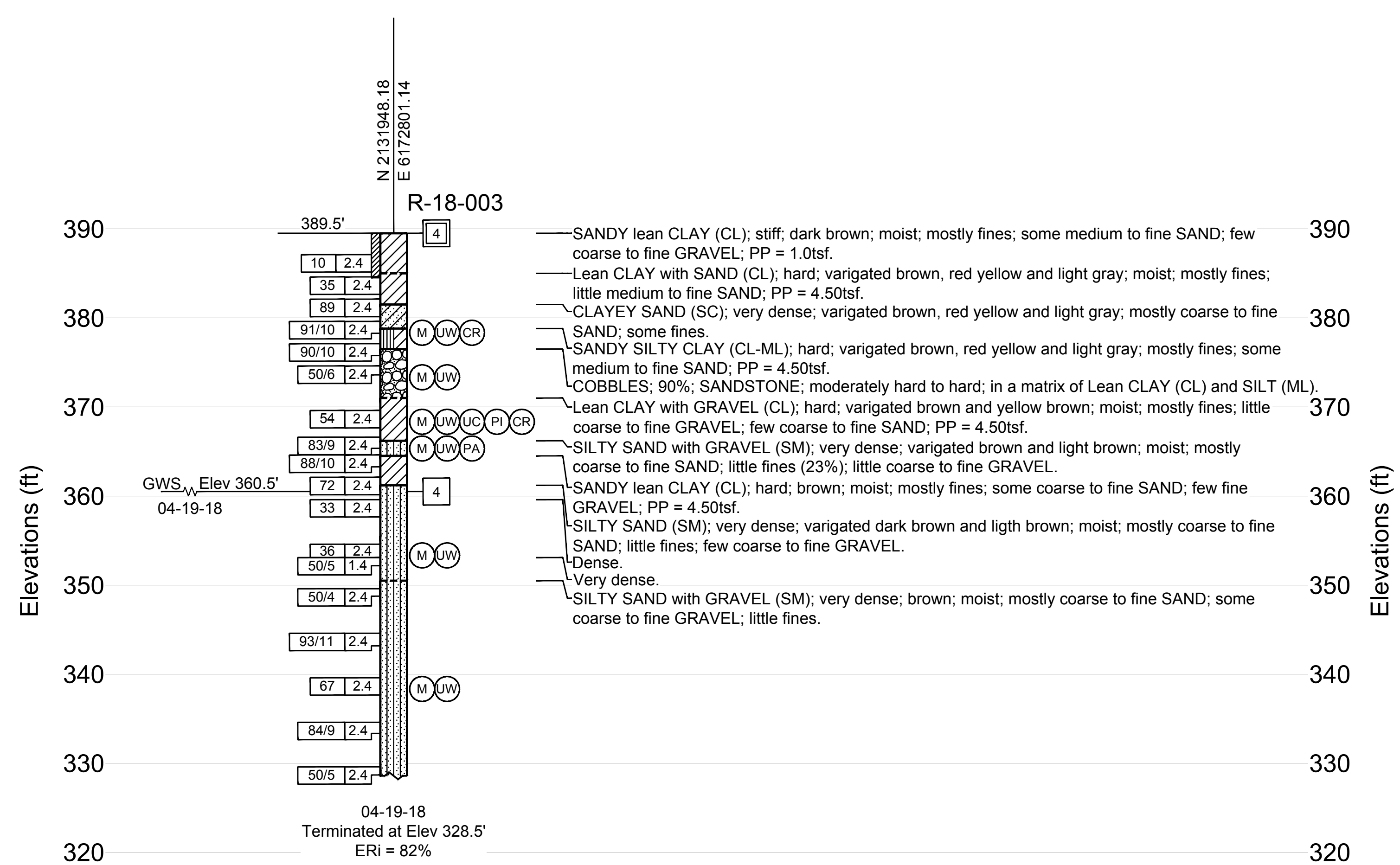
Sacramento, CA 95831 (916) 455-4225

NOTES:
1. Field classification of soils and this LOTB sheet were prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
2. Soil legends are provided in 2018 Caltrans Standard Plans A10F, A10G, and A10H.
3. Standard Penetration tests were performed in accordance with ASTM D 1586 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with steel liners.
4. "2.4 inch sampler": ID=2.4 inch, OD=2.9 inch. Driven in same manner as SPT ("1.4 inch") sampler.
5. 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.
6. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 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.
7. Blow counts shown as "REF" where less than 0.5 feet of penetration were achieved in the first seating interval.
8. Consistency and density of soils shown in ( ) where estimated.
9. 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.
10. Groundwater elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.



PROFILE
HORIZ. SCALE: NO SCALE
VERT. SCALE: 1" = 10'

Jim Foster
DESIGN OVERSIGHT
X
SIGN OFF DATE

	BY	CHECKED
DRAWING	B. Updegraff	K. Lewis
DETAILS	B. Maechler	C. Hughes
FIELD INV.	M. Gutierrez	x

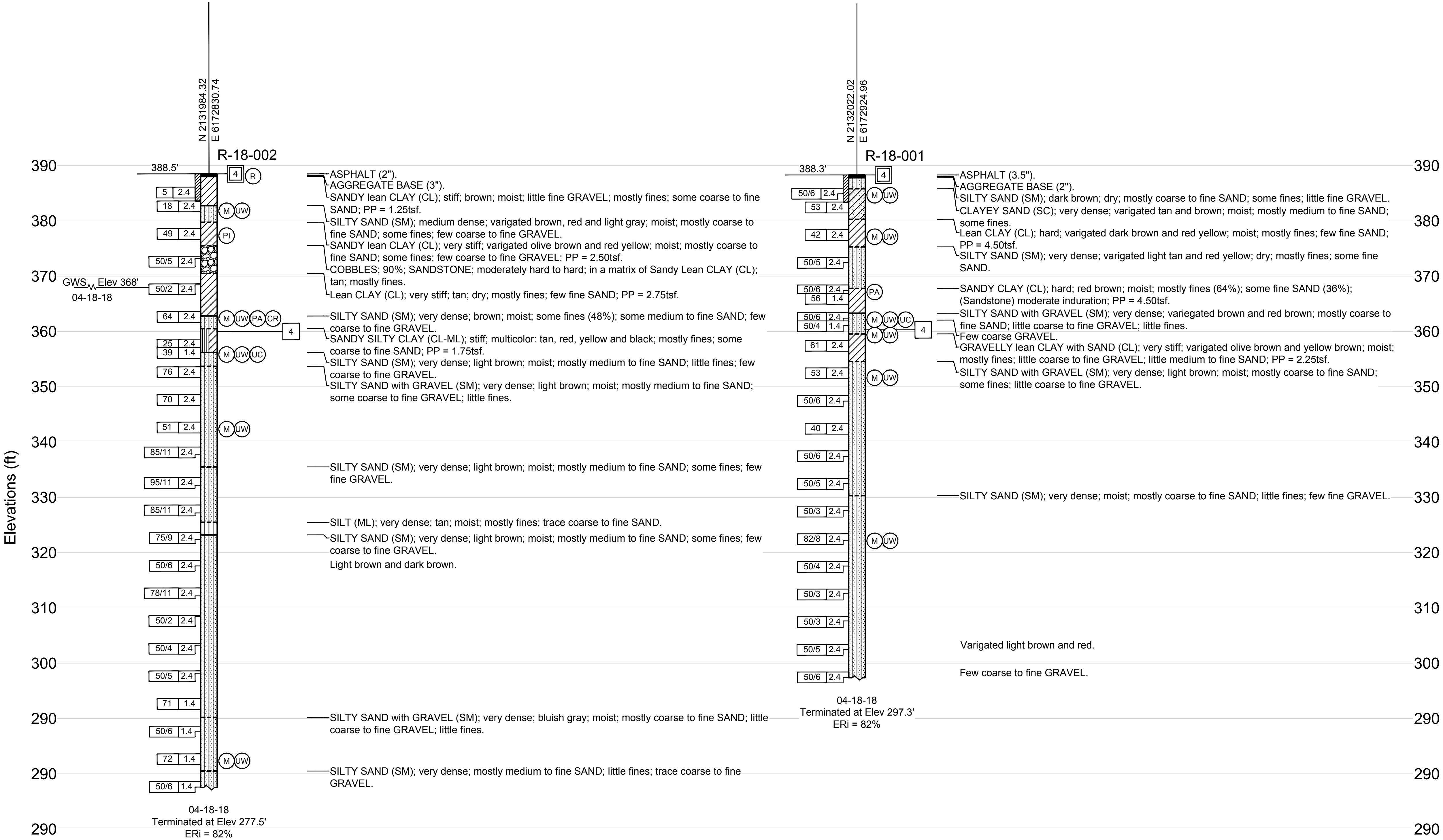
PREPARED FOR THE
MENDOCINO COUNTY
DEPARTMENT OF TRANSPORTATION

Carl Hughes
PROJECT ENGINEER

BRIDGE NO. 10C-01XX
POST MILE x

ROBINSON CREEK BRIDGE
LOG OF TEST BORINGS 1 OF 2

DESIGN DETAIL SHEET (ENGLISH) (REV.03/14/12)
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3
UNIT: X
PROJECT NUMBER & PHASE: X
CONTRACT NO.: X
DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES x
SHEET BR15 OF BR16
FILE => $REQUEST
USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME



PROFILE

HORIZ. SCALE: NO SCALE
VERT. SCALE: 1" = 10'

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
01	Men	X	X	33	33

Keiko Lewis 1/19/22
REGISTERED CIVIL ENGINEER DATE

NOT FOR CONSTRUCTION
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER KEIKO K. LEWIS No. C89543 CIVIL STATE OF CALIFORNIA

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NOTES:
1. Field classification of soils and this LOTB sheet were prepared in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010).
2. Soil legends are provided in 2018 Caltrans Standard Plans A10F, A10G, and A10H.
3. Standard Penetration tests were performed in accordance with ASTM D 1586 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven with steel liners.
4. "2.4 inch sampler": ID=2.4 inch, OD=2.9 inch. Driven in same manner as SPT ("1.4 inch") sampler.
5. 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.
6. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with the Caltrans Soil & Logging, Classification, and Presentation Manual (June 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.
7. Blow counts shown as "REF" where less than 0.5 feet of penetration were achieved in the first seating interval.
8. Consistency and density of soils shown in ( ) where estimated.
9. 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.
10. Groundwater elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
11. Groundwater was not established in boring R-18-001 due to drilling method.

Jim Foster
DESIGN OVERSIGHT
X
SIGN OFF DATE

	BY	CHECKED
DRAWING	B. Updegraff	K. Lewis
DETAILS	B. Maechler	C. Hughes
FIELD INV.	M. Gutierrez	X

PREPARED FOR THE MENDOCINO COUNTY DEPARTMENT OF TRANSPORTATION

Carl Hughes
PROJECT ENGINEER

BRIDGE NO. 10C-01XX
POST MILE X

ROBINSON CREEK BRIDGE
LOG OF TEST BORINGS 2 OF 2

DESIGN DETAIL SHEET (ENGLISH) (REV.03/14/12)
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3
UNIT: X
PROJECT NUMBER & PHASE: X
CONTRACT NO.: X
DISREGARD PRINTS BEARING EARLIER REVISION DATES X
REVISION DATES
SHEET BR16 OF BR16
FILE => $REQUEST
USERNAME => $USER DATE PLOTTED => $DATE TIME PLOTTED => $TIME

APPENDIX III

Laboratory Test Results

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

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: KE/CAP/AC

MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-001-1A	A-18-001-3B	A-18-001-6D	A-18-001-6B	A-18-001-8A
USCS Symbol	SM/SC	CL	SM	SM	SM
Depth (ft.)	3	10.5	25	26	36
Sample Length (in.)	5.577	5.963	4.135	2.969	6.007
Diameter (in.)	2.371	2.350	2.406	1.418	2.423
Sample Volume (ft^3)	0.01425	0.01496	0.01088	0.00271	0.01603
Total Mass Soil+Tube (g)	954.6	983.6	891.3	163.4	1269.8
Mass of Tube (g)	282.7	279.8	256.3	0.0	229.5
Tare No.	H20	G6	D3	C19	A16
Tare (g)	13.4	13.4	13.8	13.9	13.7
Wet Soil + Tare (g)	63.7	63.2	68.9	96.6	71.1
Dry Soil + Tare (g)	59.7	59.3	64.0	89.6	65.3
Dry Soil (g)	46.4	45.9	50.3	75.7	51.6
Water (g)	4.0	3.8	4.9	7.1	5.7
Moisture (%)	**8.7**	**8.3**	**9.7**	**9.4**	**11.1**
Dry Density (pcf)	**95.6**	**95.7**	**117.2**	**121.4**	**128.8**

Notes:

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

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: KE/CAP/AC

MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-001-14B	A-18-002-2A	A-18-002-6B	A-18-002-7B	A-18-002-7A
USCS Symbol	SM	SM	SM	SM	SM
Depth (ft.)	65.5	6	25.5	32	32.5
Sample Length (in.)	5.144	4.387	5.254	3.596	2.437
Diameter (in.)	2.392	2.304	2.385	1.421	1.414
Sample Volume (ft^3)	0.01338	0.01059	0.01358	0.00330	0.00221
Total Mass Soil+Tube (g)	1060.0	957.6	1137.4	201.3	137.8
Mass of Tube (g)	254.1	276.4	276.7	0.0	0.0
Tare No.	G4	F6	P2	G9	G19
Tare (g)	13.5	13.6	126.3	13.6	20.8
Wet Soil + Tare (g)	70.1	66.6	410.2	88.6	87.0
Dry Soil + Tare (g)	61.1	61.7	386.9	81.9	80.3
Dry Soil (g)	47.6	48.1	260.6	68.3	59.5
Water (g)	9.0	4.9	23.3	6.8	6.7
Moisture (%)	**18.9**	**10.2**	**8.9**	**9.9**	**11.2**
Dry Density (pcf)	**111.7**	**128.7**	**128.2**	**122.4**	**123.3**

Notes:

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

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: KE/CAP/AC

MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-002-10B	A-18-002-22B	A-18-003-4B	A-18-003-6A	A-18-003-7A
USCS Symbol	SM	SM	SC	SP-SM	CL
Depth (ft.)	45.5	105.5	10.5	15.5	20.5
Sample Length (in.)	3.607	5.044	5.622	3.962	5.677
Diameter (in.)	2.388	2.364	2.364	2.368	2.382
Sample Volume (ft^3)	0.00935	0.01281	0.01428	0.01010	0.01464
Total Mass Soil+Tube (g)	841.2	1048.9	1099.8	894.2	901.2
Mass of Tube (g)	258.0	273.5	211.1	275.5	0.0
Tare No.	G24	C20	H21	H8	G23
Tare (g)	20.7	13.6	13.4	13.4	13.5
Wet Soil + Tare (g)	92.6	87.7	73.1	86.1	56.7
Dry Soil + Tare (g)	83.2	74.7	66.8	81.2	52.3
Dry Soil (g)	62.4	61.1	53.4	67.8	38.9
Water (g)	9.4	13.0	6.3	4.9	4.4
Moisture (%)	**15.1**	**21.3**	**11.7**	**7.3**	**11.2**
Dry Density (pcf)	**119.5**	**110.0**	**122.8**	**125.9**	**122.0**

Notes:

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

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: KE/CAP/AC

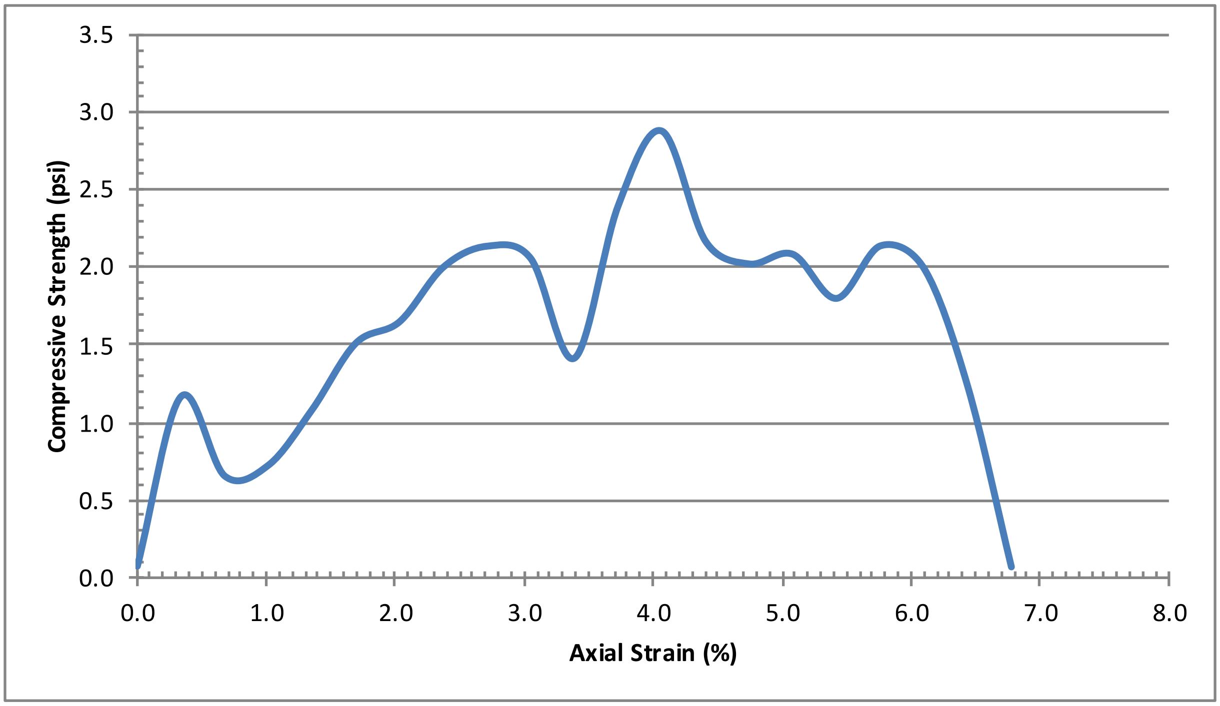
MOISTURE-DENSITY TESTS - D2216

	1	2	3	4	5
Sample No.	A-18-003-8A	A-18-003-12D	A-18-003-15B		
USCS Symbol	SM	SM	SM		
Depth (ft.)	23	35.5	50.5		
Sample Length (in.)	3.601	4.472	3.477		
Diameter (in.)	2.404	2.367	2.357		
Sample Volume (ft^3)	0.00946	0.01139	0.00878		
Total Mass Soil+Tube (g)	827.9	992.4	853.0		
Mass of Tube (g)	281.5	277.4	278.4		
Tare No.	R1	C19	D19		
Tare (g)	126.9	13.9	13.7		
Wet Soil + Tare (g)	494.7	90.9	84.3		
Dry Soil + Tare (g)	465.0	82.2	74.9		
Dry Soil (g)	338.1	68.3	61.2		
Water (g)	29.7	8.7	9.4		
Moisture (%)	**8.8**	**12.8**	**15.4**		
Dry Density (pcf)	**117.1**	**122.7**	**125.0**		

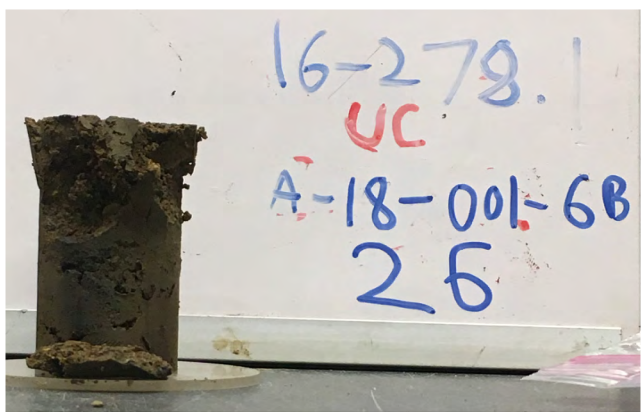
Notes:

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/13/18
Technician: CAP
Sample ID: A-18-001-6B Depth (ft): 26.0
USCS Classification: Silty SAND (SM)

UNCONFINED COMPRESSION TEST - D2166



Dry Density (pcf)	**121.4**
Water Content (%)	**9.4**
Unconfined Compressive Strength (psi)	**2.9**
Unconfined Compressive Strength (psf)	**418**
Shear Strength (psf)	**208.8**
Average Height (in)	2.969
Average Diameter (in)	1.418
Rate of strain (%)	1.0
Strain at Failure (%)	4.1

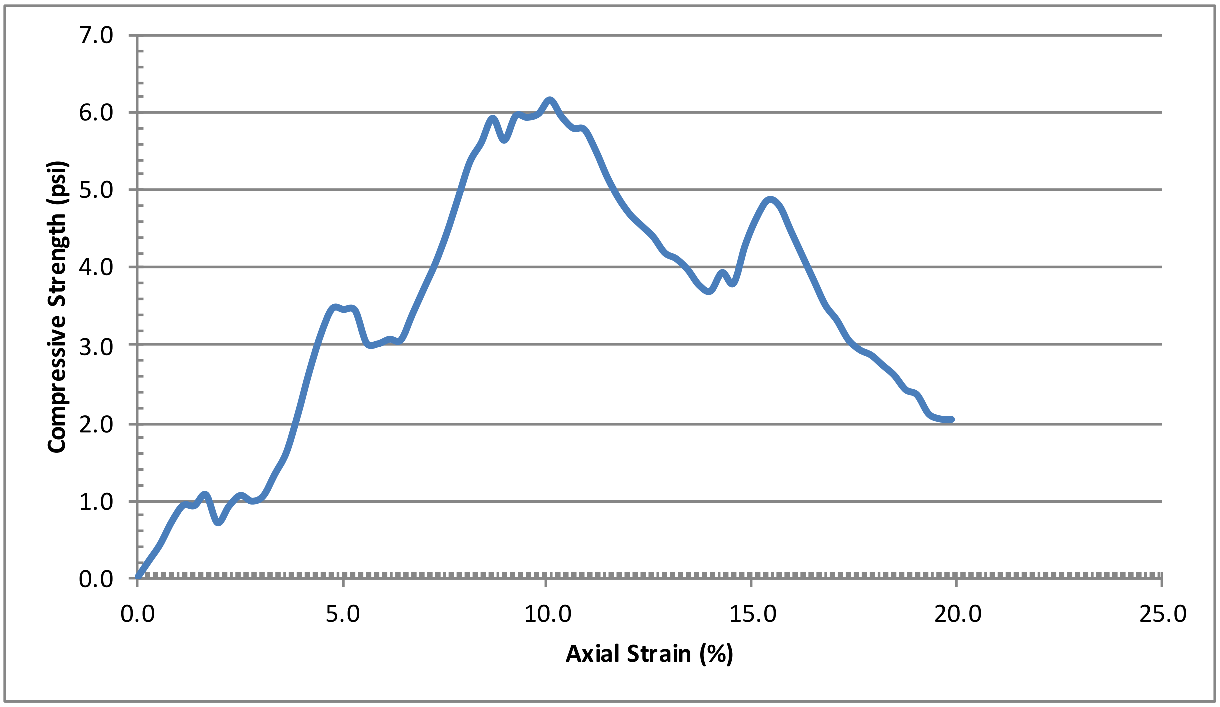


Notes:

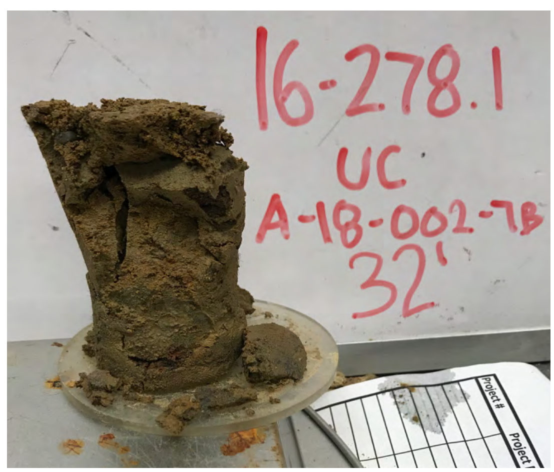
Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: CAP
Sample ID: A-18-002-7B Depth (ft): 32.0
USCS Classification: Silty SAND (SM)

UNCONFINED COMPRESSION TEST - D2166



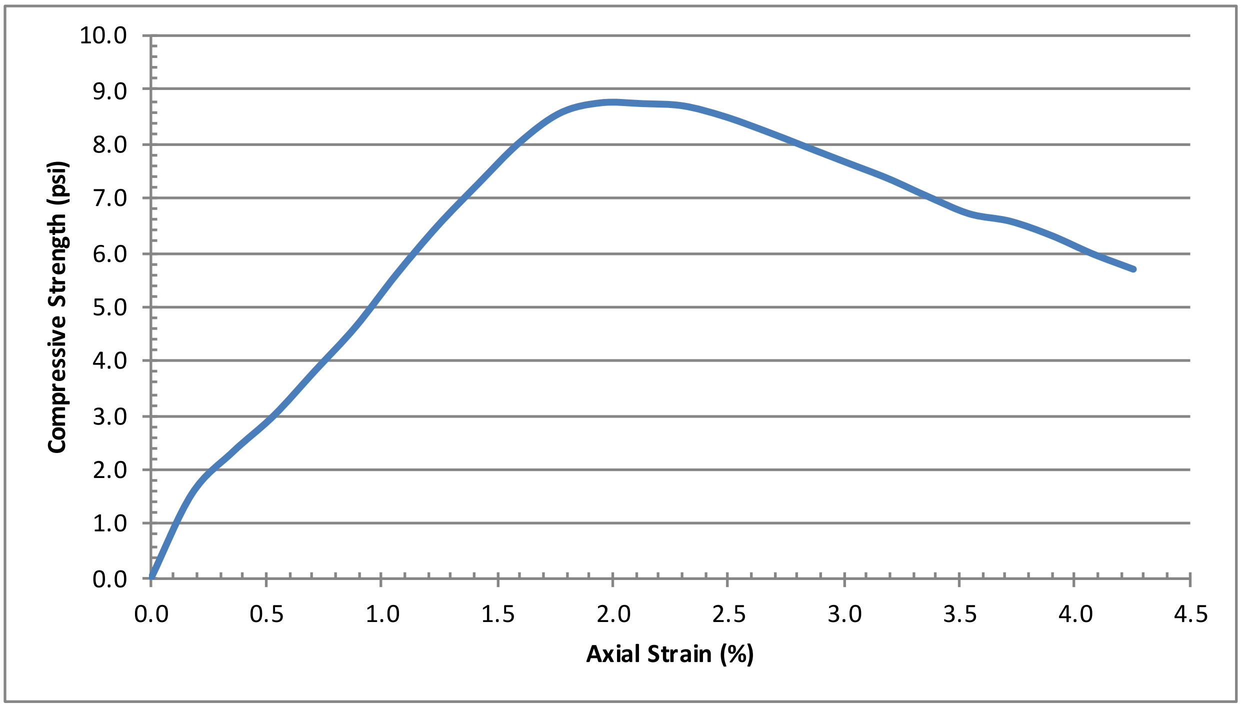
Dry Density (pcf)	**122.4**
Water Content (%)	**9.9**
Unconfined Compressive Strength (psi)	**6.2**
Unconfined Compressive Strength (psf)	**893**
Shear Strength (psf)	**446.4**
Average Height (in)	3.596
Average Diameter (in)	1.421
Rate of strain (%)	0.5
Strain at Failure (%)	10.1



Notes:

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: HFW
Sample ID: A-18-003-7B Depth (ft): 20.5
USCS Classification: Lean CLAY with GRAVEL (CL)

UNCONFINED COMPRESSION TEST - D2166

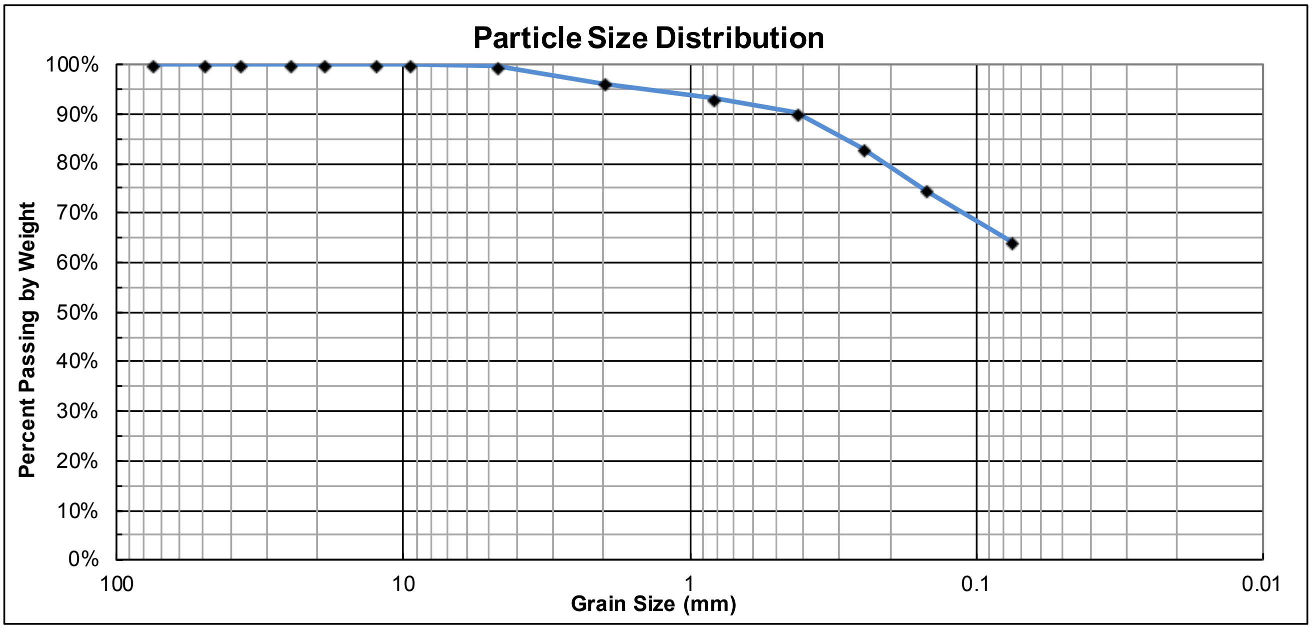


Dry Density (pcf)	**122.1**
Water Content (%)	**11.2**
Unconfined Compressive Strength (psi)	**8.8**
Unconfined Compressive Strength (psf)	**1267**
Shear Strength (psf)	**633.6**
Average Height (in)	5.677
Average Diameter (in)	2.382
Rate of strain (%)	1.0
Strain at Failure (%)	1.9

Notes:

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/18/18
Technician: GL
Sample ID: A-18-001-5A
Depth (ft): 20
USCS Classification: Sandy Lean CLAY (CL)

ASTM 6913 - Method A



% Cobble	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
	0	0	4	6	26	
0	**0**		**36**			**64**

		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	0.0	100%
	Fine	**1/2"**	12.5	0.0	100%
		3/8"	9.50	0.0	100%
		#4	4.75	1.1	100%
Sand	Coarse	**#10**	2.00	8.3	96%
	Medium	**#20**	0.825	15.3	93%
		#40	0.425	21.5	90%
	Fine	**#60**	0.250	37.4	83%
		#100	0.150	56.3	74%
		#200	0.075	79.0	64%

Coefficient of Uniformity	Coefficient of Curvature
Cu = NA	Cc = NA

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

Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/7/18
Technician: AC

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

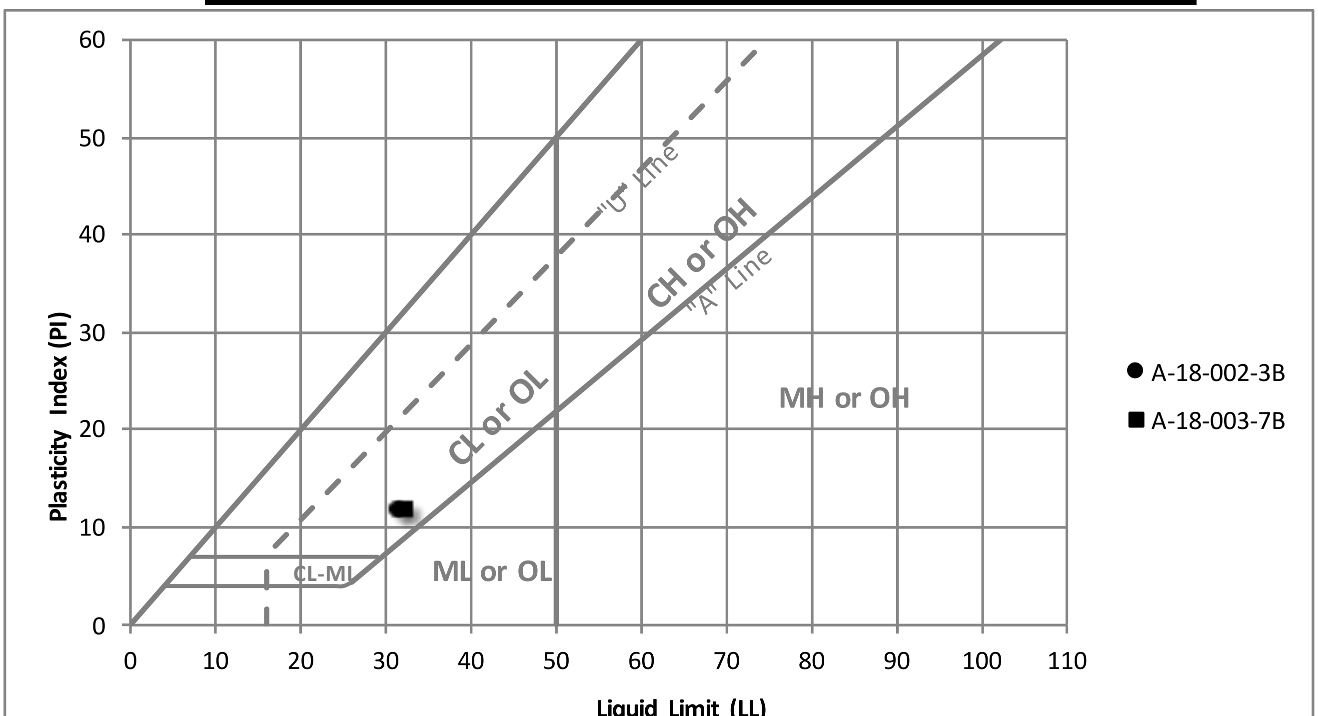
Sample No.	A-18-002-6B	A-18-003-8A			
USCS Symbol	SM	SM			
Depth (ft.)	25.5	23			
Tare No.	P2	R1			
Tare (g)	126.3	126.9			
Dry Soil + Tare (g)	386.9	465			
Dry Mass before (g)	260.6	338.1			
Dry Mass after (g)	135.2	259.2			
Percent Fines (%)	48	23			

Notes:

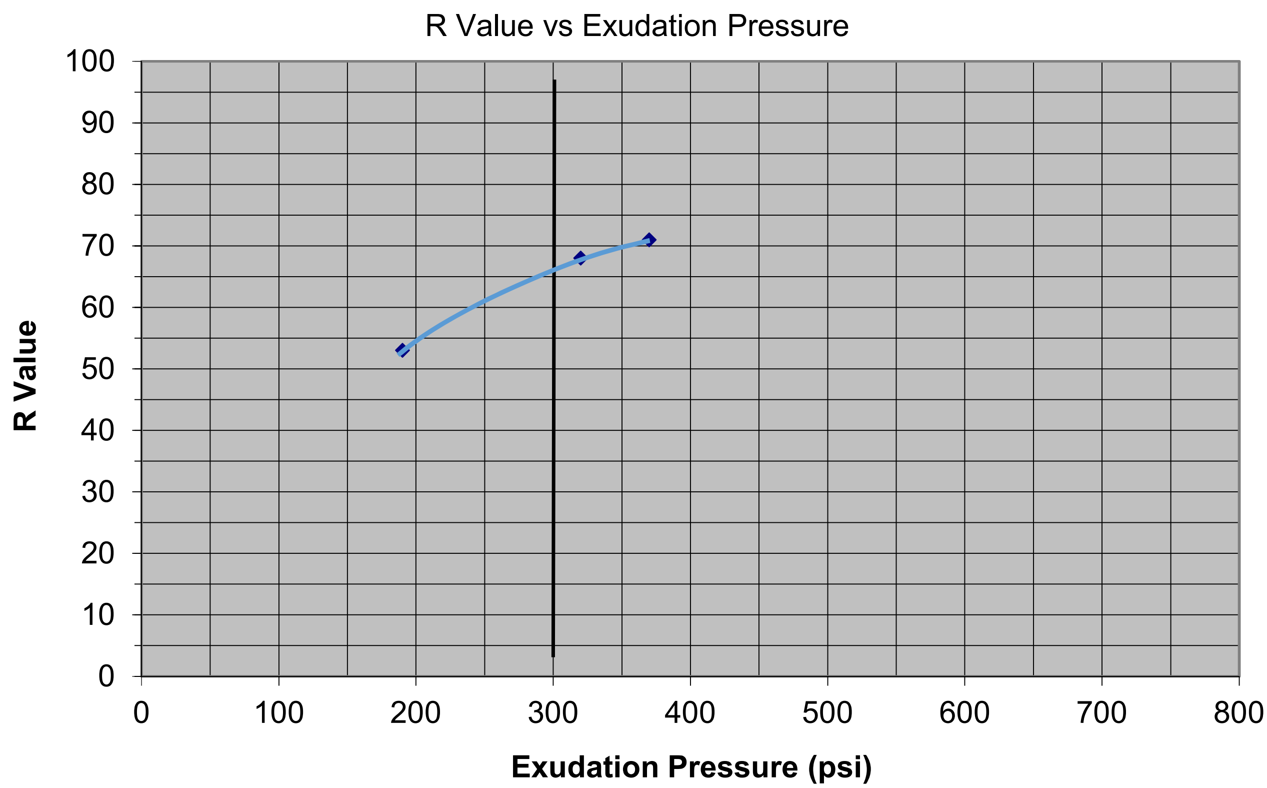
Project Name: Robinson Creek Bridge Replacement Lambert Ln
CAInc File No: 16-278.1
Date: 6/19/18
Technician: CAP

Plastic Index - ASTM D4318

Sample ID	Depth (ft)	Liquid Limit	Plastic Limit	PI
A-18-002-3B	11	31	19	12
A-18-003-7B	20.5	32	20	12



Plasticity Chart



Sample ID & Description	
Boring Number	A-18-002
Sample Depth (feet)	--
Material Description	Yellowish brown Silty SAND

Test Data			
Specimen	2091	2105	2110
Exudation Pressure (psi)	190	370	320
Expansion Dial (.0001")	18	25	21
Expansion Pressure (psf)	77.9	108.3	90.9
Resistance 'R' Value	53	71	68
Moisture at test (%)	10.8	10.4	10.6
Dry density at test (pcf)	117.3	118.8	119.7
R Value at 300 psi exudation pressure		**66**	
R Value by expansion pressure (TI=5.0)		**66**	
R Value by Equillibrium		**66**	

Geocon Consultants, Inc.
3160 Gold Valley Drive, Suite 800
Rancho Cordova, California 95742
Telephone: (916) 852-9118
Fax: (916) 852-9132

Resistance "R" Value, ASTM D2844, CTM 301

Project: Crawford 16-278.1, Robinson Creek
Location:
Number: S9763-05-125
Figure:

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/13/18
Date Submitted 06/07/18

To: Hailey Wagenman
Crawford and Associates Inc.
4020 Rocklin Rd, Ste 1
Rocklin, CA, 95677

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

The reported analysis was requested for the following:
Location : 16-278.1 ROBINSON CR Site ID: A18002-6B@25.5F
Thank you for your business.

* For future reference to this analysis please use SUN # 77171 - 161058

EVALUATION FOR SOIL CORROSION

Soil pH	7.66		
Minimum Resistivity	4.29	ohm-cm (x1000)	
Chloride	1.5 ppm	0.0002	%
Sulfate-S	3.6 ppm	0.0004	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/13/18
Date Submitted 06/07/18

To: Hailey Wagenman
Crawford and Associates Inc.
4020 Rocklin Rd, Ste 1
Rocklin, CA, 95677

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

The reported analysis was requested for the following:
Location : 16-278.1 ROBINSON CR Site ID: A18003-4B@10.5F
Thank you for your business.

* For future reference to this analysis please use SUN # 77171 - 161056

EVALUATION FOR SOIL CORROSION

Soil pH	7.46	
Minimum Resistivity	2.95	ohm-cm (x1000)
Chloride	6.1 ppm	0.0006 %
Sulfate-S	12.2 ppm	0.0012 %

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 06/13/18
Date Submitted 06/07/18

To: Hailey Wagenman
Crawford and Associates Inc.
4020 Rocklin Rd, Ste 1
Rocklin, CA, 95677

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

The reported analysis was requested for the following:
Location : 16-278.1 ROBINSON CR Site ID: A18003-7B@20.5F
Thank you for your business.

* For future reference to this analysis please use SUN # 77171 - 161057

EVALUATION FOR SOIL CORROSION

Soil pH	7.10		
Minimum Resistivity	4.56	ohm-cm (x1000)	
Chloride	1.3 ppm	0.0001	%
Sulfate-S	3.7 ppm	0.0004	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

APPENDIX IV

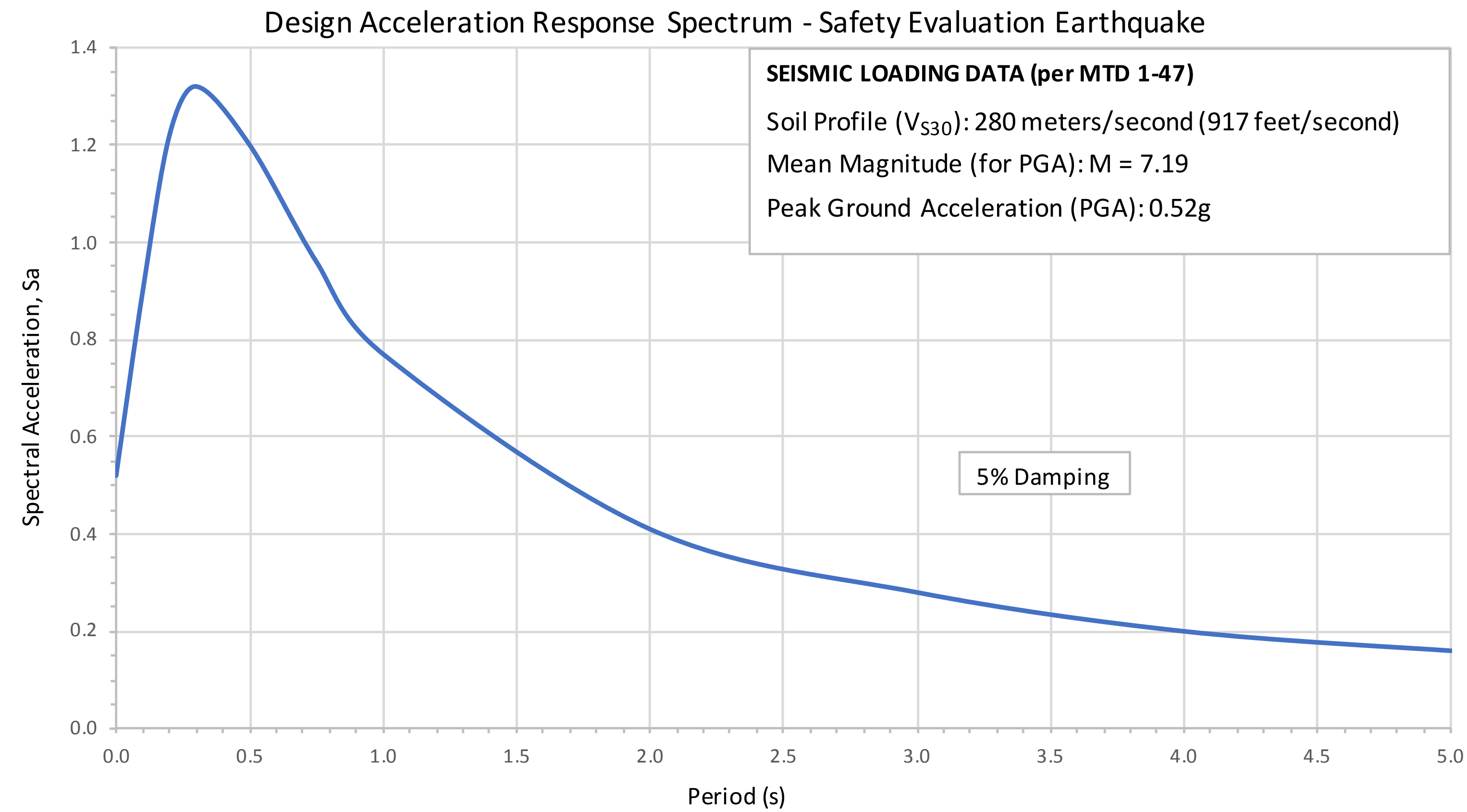
Ground Motion Data Sheet

GROUND MOTION DATA SHEET - SAFETY EVALUATION EARTHQUAKE (SEE)

Replacement of Robinson Bridge at Lambert Lane (Bridge No. 10C-0146
Mendocino County, California

Caltrans Seismic Design Criteria: V2.0
Caltrans ARS Online Version: V3.0.2
Date Accessed: 10/14/2021

Period (s)	Spectral Acceleration, Sa (g)
0.000	0.520
0.100	0.900
0.200	1.220
0.300	1.320
0.500	1.200
0.750	0.960
1.000	0.770
2.000	0.410
3.000	0.280
4.000	0.200
5.000	0.160



The Accleration Response Spectrum (ARS) for the Safety Evaluation Earthquake (SEE) is based on the USGS 2014 National Seismic Hazard Map for 975-year return period, (Hazard Model/Edition "Dynamic Conterminous U.S. 2014 (Update)(V4.20)") hazard data obtained by using ARS Online. Modifications for basin-effects and/or near-fault effects were applied, where applicable.

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

Taber
Since 1954

Site Latitude: 39.0084
Site Longitude: -123.3683

Crawford File No: 16-278.1

APPENDIX IV

APPENDIX V

Foundation Design Calculations

This appendix presents our foundation design calculations that include geotechnical design parameters, assumptions, methodology, and summaries the results for shear wave velocity (V_{S30}), deep foundations (Bridge Abutments - pile compression resistance, tension resistance, lateral resistance, settlement, negative skin friction and group reduction) and shallow foundations (Retaining Wall - bearing resistance and settlement). Our pile foundation analysis and recommendations are in accordance with the AASHTO LRFD Bridge Design Specifications (8th Edition) with Caltrans Amendments.

The contents of this appendix are presented in the following order:

GEOTECHNICAL DESIGN PARAMETERS

The idealized geotechnical engineering properties and strength/bearing characteristics of foundation materials selected for use in this report have been derived/established from a combination of:

- Visual logging of earth materials and drilling procedures by a staff engineer;
- Earth materials classification based on laboratory test results (as applicable);
- Unit weight values based on laboratory test results and/or published correlations;
- Friction angles based on published blow count correlations;
- Undrained shear strength (cohesion) values based on unconfined compressive strength test results, pocket penetrometer data and/or published blow count correlations;
- Average N_{SPT} values recorded in the soil borings and corrected for hammer efficiency and overburden pressure (as applicable);
- Design groundwater at elevation 368.0 ft; and
- Engineering experience and judgment based on past projects with a similar geologic environment/profile.

The idealized geotechnical parameters used in our analysis are shown in Table V-1 through V-3.

Table V-1: Idealized Geotechnical Parameters – Abutment 1 (R-18-002)

Elevation (ft)	Soil Description	N_{60}	Soil Type		Unit Weight (lb/ft^3)	Friction Angle (degrees)	Cohesion (psf)	Strain Factor, E50 (dim.)	p-y Modulus, k (lb/in^3)
			Axial Capacity	L-Pile					
380.0 to 368.0	Cobbles with Sandy Lean Clay	55	Sand	Sand (Reese)	125	36	---	---	225
368.0 to 365.5	Very Dense Silty Sand	55	Sand	Sand (Reese)	62	36	---	---	125
365.5 to 340.5	Very Dense Silty Sand with Gravel	36	Sand	Sand (Reese)	67	35	--	--	125
340.5 to 310.5	Very Dense Silty Sand and Silt	58	Sand	Sand (Reese)	72	38	--	--	125
310.5 to 278.0	Very Dense Silty Sand with Gravel	>70	Sand	Sand (Reese)	72	40	--	--	125

Notes: Elevations are based on project datum.
Use buoyant unit weight as shown below design groundwater (elev. 368.0 ft)

Table V-2: Idealized Geotechnical Parameters – Abutment 2 (R-18-001)

Elevation (ft)	Soil Description	N_{60}	Soil Type		Unit Weight (lb/ft^3)	Friction Angle (degrees)	Cohesion (psf)	Strain Factor, E50 (dim.)	p-y Modulus, k (lb/in^3)
			Axial Capacity	L-Pile					
376.0 to 368.0	Very Dense Silty Sand	60	Sand	Sand (Reese)	125	38	---	---	225
368.0 to 361.0	Very Dense Silty Sand with Gravel	60	Sand	Sand (Reese)	62	38	---	---	125
361.0 to 356.0	Very Stiff Gravelly Lean Clay with Sand	54	Clay	Stiff Clay w/o Free Water (Reese)	62	--	2,200	0.006	--[1]
356.0 to 341.0	Very Dense Silty Sand with Gravel	36	Sand	Sand (Reese)	70	35	--	--	125
341.0 to 298.0	Very Dense Silty Sand	70	Sand	Sand (Reese)	72	38	--	--	125

Notes: Elevations are based on project datum.
Use buoyant unit weight as shown in table below design groundwater (elev. 368.0 ft)
[1]LPILE does not report the k value for Clay without Free Water soil type.
The k value is internally computed using the cohesion and E50 parameters.

Table V-3: Idealized Geotechnical Parameters – Retaining Wall 1 (R-18-003)

Elevation (ft)	Soil Description	N_{60}	Soil Type		Unit Weight (lb/ft^3)	Friction Angle (degrees)	Cohesion (psf)	Strain Factor, E50 (dim.)	p-y Modulus, k (lb/in^3)
			Axial Capacity	L-Pile					
389.0 to 377.0	Stiff to Hard Lean Clay with Sand	30	Clay	Stiff Clay w/o Free Water (Reese)	125	--	3,500	0.005	--[1]
377.0 to 371.0	Cobbles with Clay and Silt	52	Sand	Sand (Reese)	130	36	---	---	225
371.0 to 368.0	Hard Lean Clay with Gravel	48	Clay	Stiff Clay w/o Free Water (Reese)	125	--	4,000	0.005	--
368.0 to 361.0	Hard Sandy Lean Clay	63	Clay	Stiff Clay w/o Free Water (Reese)	62	--	4,000	0.005	--
361.0 to 351.0	Very Dense Silty Sand	26	Sand	Sand (Reese)	67	35	--	--	125
351.0 to 328.5	Very Dense Silty Sand with Gravel	56	Sand	Sand (Reese)	72	38	--	--	125

Notes: Elevations are based on project datum.
Use buoyant unit weight shown in table below design groundwater (elev. 368.0 ft)
[1]LPILE does not report the k value for Clay without Free Water soil type.
The k value is internally computed using the cohesion and E50 parameters.

SHEAR WAVE VELOCITY

A correlated shear wave velocity (V_{S30}) in the upper 30 meters (100 ft) of the soil profile of each boring completed for this project (R-18-001, R-18-002 and R-18-003) was determined based on correlations with SPT N-values corrected for hammer efficiency (N_{60}) using the equations outlined by Caltrans[1]. For a non-standard sampler (i.e., non-SPT sampler), the in-situ N-value was corrected to an *Equivalent SPT N-value* using guidance by Caltrans[2], then adjusted to provide an *Equivalent N_{60}* value.

The recommended V_{S30} of 280 meters per second (about 917 ft/sec) is is approximately the lowest calculated V_{S30} of all the borings completed for this project. This value corresponds to a "stiff soil" with 180 m/s < Vs < 360 m/s for the upper 100 ft of the soil profile. For our evaluation, we used latitude 39.0084°N and longitude 123.3683°W for the site coordinates.

Shear wave velocity calculations (input data and output results) for the individual borings are included herein.

[1] California Department of Transportation, *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations*, Appendix A, November 2012.
[2] Caltrans Geotechnical Manual, Sampler Size Conversions to SPT N-value, Soil Correlations Module (March 2021).

DEEP FOUNDATIONS (BRIDGE ABUTMENTS)

Recommendations are summarized below for 30-inch cast-in-drilled-hole (CIDH) piles at Abutment 1 and 2. Refer to Section 11 of the Foundation Report that summarizes the foundation data and loading conditions provided by the designer that were used in our pile analysis.

COMPRESSIVE RESISTANCE

The side (compressive) resistance for the 30-inch CIDH pile foundations was evaluated using Load and Resistance Factor Design (LRFD) method and factors from AASHTO LRFD Bridge Design Specifications (8th Edition) with current Caltrans amendments (including scour). The α-Method equations (10.8.3.5.1b-1, -2 and -3) as presented in AASHTO LRFD Bridge Design Specifications (8th Edition) were used for cohesive (clay) layers. The β-Method equations (10.8.3.5.2b-1, -2 and -3) as presented in the California Amendments to AASHTO LRFD Bridge Design Specifications (8th Edition) were used for cohesionless (sand) layers.

The idealized geotechnical parameters shown in Tables V-1 and V-2 were used to calculate the design tip elevations for the recommended piles. Design groundwater was modeled at elev. 368.0 ft.

Skin friction contributions are only considered in our compressive resistance analysis. For our foundation design analysis, the top 5 ft of the pile and the bottom 3 ft of pile (slightly more than one shaft diameter) are excluded from contributing to geotechnical resistance. Tip resistance in axial compression was neglected in consideration of slurry installation method, consistent with current Caltrans guidelines for CIDH pile design.

To determine the required nominal resistance in axial compression of the CIDH piles for the controlling limit state we used the foundation data provided by QEI and compared the factored strength limit and factored extreme limit loads by dividing each by their respective geotechnical resistance factor in side resistance (φ_{qs} = 0.7 for strength limit; φ_{qs} = 1.0 for extreme limit) consistent with current AASHTO LRFD Bridge Design Specifications and California amendments. No compression demand at the Extreme Limit State load is indicated for the 30-inch CIDH piles. Therefore, the Factored Strength Limit Load is the controlling load requirement.

Refer to the input data sheets and CIDH Pile Nominal Resistance in Side Friction graphs in this appendix that show the nominal resistance in side friction vs. pile tip elevation for the planned 30-inch diameter CIDH piles.

TENSION (UPLIFT) RESISTANCE

No tension demands are indicated for the 30-inch CIDH pile foundations.

LATERAL RESISTANCE

QEI requested that Crawford provide LPILE parameters for use in lateral pile capacity analysis. The idealized geotechnical parameters in this appendix present the recommended soil information to evaluate the lateral capacity.

The LPILE computer program, developed by Ensoft, is used to estimate lateral pile design loads for vertical piles which would provide approximately ¼-inch and 1-inch pile head deflection assuming a pinned (free-head) or fixed-head condition. We understand QEI will be performing the lateral pile analysis for new bridge foundation elements.

For multiple row shading apply appropriate P-Multipliers (Pm) shown in Table V-4 consistent with Section 10.7.2.4 of the April 2019 California Amendments to AASHTO LRFD Bridge Design Specifications (8th Edition). Interpolate the P-Multiplier value for intermediate pile spacing. For loading direction and spacing refer to Figure 10.7.2.4-1 of AASHTO LRFD Bridge Design Specifications (8th Edition).

Table V-4: Pile P-Multipliers, P_m for Multiple Row Shading

Pile CTC spacing (in the Direction of Loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
2.0B	0.60	0.35	0.25
3.0B	0.75	0.55	0.40
5.0B	1.0	0.85	0.70
7.0B	1.0	1.0	0.90

CTC = Center-To-Center Spacing / B = Pile Diameter / Use a P-Multiplier = 1.0 for Pile CTC Spacing ≥ 8B

Where the loading direction is perpendicular to a single row of piles use a P-Multiplier of 0.80, 0.90, and 1.0 for pile spacing of 2.5B, 3B and ≥4B, respectively.

Lateral pile analysis should also incorporate change in foundation due to limit state for scour in accordance with the latest California Amendments to Section 3.7.5 of AASHTO LRFD Bridge Design Specifications (8th Edition).

NEGATIVE SKIN FRICTION

Negative skin friction is not anticipated to develop for CIDH piles at the abutments.

SETTLEMENT

Based on the subsurface data obtained for this study, total pile settlement at each support under service load is estimated to be within the permissible 2.0-inch (likely < 1-inch) settlement for the recommended pile foundations. Since the piles will be embedded adequately into dense/hard soils, and the piles will not be subjected to downdrag loads, a detailed assessment of the pile group settlement is not considered warranted.

PILE GROUP REDUCTION

The foundation soils predominately consist of very stiff to hard cohesive and medium dense to dense/very dense granular layers. The pile diameter (b) of the CIDH piles is 30-inches. There are two rows of piles at each abutment and the pile center-to-center spacing is slightly greater than 2.5 times the pile diameter (about 2.56b) and the bottom of the pile cap will be above the design scour elevation.

According to the AASHTO LRFD BDS 8th Edition (Section10.7.3.9 – Resistance of Pile Groups in Compression) with California amendments:

- For Cohesive Soil - If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.
- For Granular Soil – The efficiency factor (η) shall be determined by linear interpolation for intermediate center to center pile spacing as shown in Table 10-8.3.6.3-1[3].

Therefore, a group efficiency factor (η) of 0.69 was used in our compressive resistance pile analysis.

[3] California Amendments to AASHTO LRFD Bridge Design Specifications-8th Edition.

SHALLOW FOUNDATIONS (RETAINING WALL)

Recommendations are summarized below for the Caltrans Standard Type 1A (Case 1) retaining wall. Refer to Section 11 of the Foundation Report that summarizes the retaining wall spread footing data and loading conditions presented on the Caltrans 2018 Standard Plan (SP) sheet B3-3A used in our retaining wall analysis.

BEARING RESISTANCE

The spread footing foundations for the retaining wall should be designed/constructed in accordance with the recommendations provided in Section 11.2 of the foundation report and consistent with Caltran's 2018 Standard Plan B3-3A design. The spread footing foundations are expected to meet/exceed the required gross uniform bearing stresses (Strength and Extreme State I/II Loads) without excessive settlement for the indicated net bearing stresses (Service Limit State I Load).

The nominal bearing resistance for the retaining wall spread footing established on an engineered fill prism was calculated using the theoretical method formula (Munfakh, et al. 2001) presented in Section 10.6.3.1.2 of the AASHTO LRFD BDS.

The spread footing was modeled with no embedment and groundwater at the bottom of the footing. The bearing material below the footing was modeled as a granular soil with a total unit weight = 120 pcf, internal friction angle = 34 degrees and cohesion = 0 psf.

We used a resistance factor (ϕ_b) = 0.55 at the strength limit state; 0.8 at the extreme limit state; and 1.0 at the service limit state in our analysis per AASHTO LRFD BDS Sections 11.5.7 and 11.5.8 with California amendments.

The nominal bearing resistance calculations are attached to this appendix.

SETTLEMENT

For a retaining wall spread footing constructed on/in the recommended engineered fill prism, the total post-construction soil settlement is expected to be minimal (less than 1-inch) with differential settlement on order of one-half the total settlement.

Boring Data

Sampler Size Conversions to SPT N-value

When sampler sizes vary from that of a SPT sampler (per ASTM 1586), conversions are needed to modify the field SPT N-values. For conversions to SPT sampler from other sampler sizes where weight (140 lb.) and height (18 inches) do not vary from ASTM 1586, the following table provides conversion values for common alternate sampler sizes.

Table 1. Conversion Values for Sampler Sizes

Sampler Type	Outside Diameter (inches)	Inside Diameter (inches)	Conversion to SPT N-value	
			Clay	Sand
1.41-inch	2.0	1.375	1.00	1.00
2-inch	2.5	1.95	0.85	0.63
2.4-inch	3.0	2.4	0.65	0.41
2.5-inch	3.875	2.5	0.24	0.13

R-18-002
Abutment 1
Robinson Creek Bridge at Lambert Lane

Project: **Robinson Creek Bridge at Lambert Lane**
Job No.: **16-278.1**
Date: **9/15/21**
Boring: **R-18-002**
Support: **Abutment 1**
By: **WEN**

Boring Elevation: 388.5 feet
Groundwater Depth: 20.5 feet — Groundwater Elevation: 368.0 feet
Hammer Weight: 140.0 pounds
Hammer Drop: 30.0 inches
Hammer Efficiency (ER): 82.0 % (If not known, assume ER = 60% for conventiaonal drop hammer using a rope and cathead and ER = 80% for automatic trip hammer.)

Field Data												Laboratory Test Results										Total Stress (Undrained)		Effective Stress (Drained)	
Sample Number	Sample D_o (inches)	Sample D_i (inches)	Depth of Sample (ft)	Depth to Bottom of Layer (ft)	Elevation of Sample (ft)	Soil Type	N (blows/ft)	N_{60} (blows/ft)	Pocket Pen. PP (tsf)	Torvane TV (tsf)	Vane Shear VS (tsf)	Dry Unit Weight (pcf)	Moisture Content Wn (%)	Total Unit Weight (pcf)	Average Mean Grain Size D_{50} (mm)	Percent Fines (-200)	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Liquidity Index LI	qu Unconfined Compressive Strength (tsf)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)
1	3.000	2.400	2.5	3.0	386.0	CL	5	4	1.25			120.0	0.0	120.0											
2	3.000	2.400	5.0	8.5	383.5	SM	18	10				128.2	10.2	141.0											
3	3.000	2.400	10.0	12.5	378.5	CL	49	44	2.50			128.2	10.2	141.0			31	19	12	-0.73					
4	3.000	2.400	15.0	17.5	373.5	SM	120	67				128.2	10.2	141.0											
5	3.000	2.400	20.0	23.0	368.5	CL	300	268	2.75			128.2	10.2	141.0											
6	3.000	2.400	25.0	25.5	363.5	SM	64	36				128.2	8.9	140.0		48									
7	3.000	2.400	30.0	31.5	358.5	CL	25	22	1.75			123.3	11.2	137.0											
8	2.000	1.400	31.5	35.0	357.0	SM	39	53				123.3	11.2	137.0							0.45				
9	3.000	2.400	35.0	38.0	353.5	SM	76	43				123.3	11.2	137.0											
10	3.000	2.400	40.0	43.0	348.5	SM	70	39				**119.5**	**15.1**	138.0											
11	3.000	2.400	45.0	48.0	343.5	SM	51	29				119.5	15.1	138.0											
12	3.000	2.400	49.5	53.0	339.0	SM	93	52				119.5	15.1	138.0											
13	3.000	2.400	55.0	58.0	333.5	SM	104	58				119.5	15.1	138.0											
14	3.000	2.400	60.0	63.0	328.5	SM	93	52				119.5	15.1	138.0											
15	3.000	2.400	65.0	68.0	323.5	SM	100	56				119.5	15.1	138.0											
16	3.000	2.400	70.0	73.0	318.5	SM	100	56				119.5	15.1	138.0											
17	3.000	2.400	75.0	78.0	313.5	SM	85	76				119.5	15.1	138.0											
18	3.000	2.400	80.0	83.0	308.5	SM	300	268				119.5	15.1	138.0											
19	3.000	2.400	85.0	88.0	303.5	SM	150	134				119.5	15.1	138.0											
20	3.000	2.400	90.0	93.0	298.5	SM	120	107				119.5	15.1	138.0											
21	2.000	1.400	95.0	98.0	293.5	SM	71	97				119.5	15.1	138.0											
22	2.000	1.400	100.0	103.0	288.5	SM	100	137				**110.0**	**21.3**	133.0											
23	2.000	1.400	105.0	108.0	283.5	SM	72	98				110.0	21.3	133.0											
24	2.000	1.400	110.0	110.5	278.5	SM	100	137				110.0	21.3	133.0											

NOTES

Soil Type: Enter Unified Soil Classification System Symbol.
Silt (ML) may or may not be cohesive. This worksheet models ML as a non-cohesive soil. For ML that exhibits and/or is expected to behave as a cohesive soil enter CL-ML, ML-CL, ML/CL or CL/ML for the Soil Type.
NP = Non Plastic — SPT = Standard Penetration Test
NA = Not Applicable — Liquidity Index (LI) = (Wn-PL)/(LL-PL)

Shear Wave Velocity (Vs)

R-18-002
Abutment 1
Robinson Creek Bridge at Lambert Lane

Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations", Appendix A, pgs 15-18, November 2012.

Project: **Robinson Creek Bridge at Lambert Lane**
Job No: **16-278.1**
Date: **9/15/21**
Boring: **R-18-002**
Support: **Abutment 1**

Hammer Efficiency (ER): 82.0 %

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Type	Undrained Shear Strength S_u (psf)	Rock *	N (bpf)	N_{SPT} (bpf) **	N_{60}	σ'$_v$ (ksf)	σ'$_v$ (kPa)	d Layer Thickness in upper 30 m (m)	Granular Soil Vs SAND (m/sec)	Granular Soil Vs SILT (m/sec)	Cohesive Soil Vs Cohesive[1] (m/sec)	Cohesive Soil Vs Cohesive[2] (m/sec)	Sedimentary Rock Vs (m/sec)	Profile Vs (m/sec)	Profile D/Vs (sec)
1	2.5	3.0	3.0	2.4	CL	1250		5	3	4	0.30	14.36	0.91			160	115		160	0.006
2	5.0	8.5	5.5	2.4	SM			18	7	10	0.64	30.74	1.68	160					160	0.010
3	10.0	12.5	4.0	2.4	CL	2500		49	32	44	1.35	64.49	1.22			219	253		219	0.006
4	15.0	17.5	5.0	2.4	SM			120	49	67	2.05	98.25	1.52	252					252	0.006
5	20.0	23.0	5.5	2.4	CL	2750		300	196	268	2.76	132.01	1.68			229	310		229	0.007
6	25.0	25.5	2.5	2.4	SM			64	26	36	3.18	152.22	0.76	264					264	0.003
7	30.0	31.5	6.0	2.4	CL	1750		25	16	22	3.55	170.15	1.83			186	252		186	0.010
8	31.5	35.0	3.5	1.4	SM			39	39	53	3.67	175.51	1.07	283					283	0.004
9	35.0	38.0	3.0	2.4	SM			76	31	43	3.93	188.01	0.91	282					282	0.003
10	40.0	43.0	5.0	2.4	SM			70	29	39	4.30	205.97	1.52	285					285	0.005
11	45.0	48.0	5.0	2.4	SM			51	21	29	4.68	224.06	1.52	283					283	0.005
12	49.5	53.0	5.0	2.4	SM			93	38	52	5.02	240.35	1.52	304					304	0.005
13	55.0	58.0	5.0	2.4	SM			104	43	58	5.44	260.26	1.52	313					313	0.005
14	60.0	63.0	5.0	2.4	SM			93	38	52	5.81	278.36	1.52	315					315	0.005
15	65.0	68.0	5.0	2.4	SM			100	41	56	6.19	296.46	1.52	322					322	0.005
16	70.0	73.0	5.0	2.4	SM			100	41	56	6.57	314.56	1.52	327					327	0.005
17	75.0	78.0	5.0	2.4	SM			85	56	76	6.95	332.66	1.52	341					341	0.004
18	80.0	83.0	5.0	2.4	SM			300	196	268	7.33	350.75	1.52	380					380	0.004
19	85.0	88.0	5.0	2.4	SM			150	98	134	7.70	368.85	1.52	369					369	0.004
20	90.0	93.0	5.0	2.4	SM			120	78	107	8.08	386.95	1.52	365					365	0.004
21	95.0	98.0	5.0	1.4	SM			71	71	97	8.46	405.05	1.52	365					365	0.004
22	100.0	103.0	5.0	1.4	SM			100	100	137	8.83	422.67	0.13	380					380	0.000
23	105.0	108.0	5.0	1.4	SM			72	72	98	9.18	439.57	NA	373					NA	NA
24	110.0	110.5	2.5	1.4	SM			100	100	137	9.53	456.47	NA	380					NA	NA
												Sum =	**30.00**						290	0.111

* Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
** Corrected for sample diameter

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

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

d (m)	30.00
Vs(d) (m/sec)	270
V_{S30} (m/sec)	**290**
V_{S30} (ft/sec)	**952**

Soil Profile Type
'D' (180 m/s < Vs < 360 m/s)

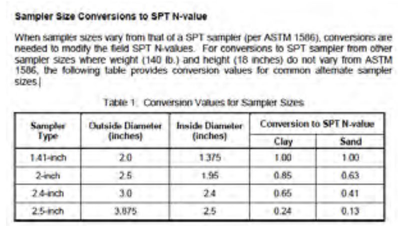
Shear Wave Velocity Correlations

Sand: $Vs = \exp[4.045 + 0.096(\ln(N_{60})) + 0.236(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec
Silt: $Vs = \exp[3.738 + 0.178(\ln(N_{60})) + 0.231(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec
Cohesive[1]: $Vs = 203(S_u/P_a)^{0.475} \leq 310$ m/sec, valid for range of 100 to 310 m/sec
Cohesive[2]: $Vs = \exp[3.996 + 0.230(\ln(N_{60})) + 0.164(\ln(\sigma'_v))]$, valid for range of 100 to 310 m/sec & $N_{60} \geq 3$
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \leq 560$/m/sec
P_a = Atmospheric Pressure = 2116.2 psf

Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) for SAND and SILT based on SPT correlations are valid where $N_{60} \leq 100$ and $\sigma'_v \leq 506$ kPa.
4) Undrained Shear Strength (S_u) based on 0.5(Pocket Penetrometer) in psf.

Boring Data

A-18-003
Retaining Wall 1
Robinson Creek Bridge at Lambert Lane


Sampler Size Conversions to SPT N-value

When sampler sizes vary from that of a SPT sampler (per ASTM 1586), conversions are needed to modify the field SPT N-values. For conversions to SPT sampler from other sampler sizes where weight (140 lb.) and height (18 inches) do not vary from ASTM 1586, the following table provides conversion values for common alternate sampler sizes.

Table 1. Conversion Values for Sampler Sizes

Sampler Type	Outside Diameter (inches)	Inside Diameter (inches)	Conversion to SPT N-value	
			Clay	Sand
1.41-inch	2.0	1.375	1.00	1.00
2-inch	2.5	1.95	0.85	0.63
2.4-inch	3.0	2.4	0.65	0.41
2.5-inch	3.875	2.5	0.24	0.13

Project: **Robinson Creek Bridge at Lambert Lane**
Job No.: **16-278.1**
Date: **5/4/18**
Boring: **A-18-003**
Support: **Retaining Wall 1**
By: **ETT**

Boring Elevation: 387.0 feet
Groundwater Depth: 26.5 feet — Groundwater Elevation: 360.5 feet
Hammer Weight: 140.0 pounds
Hammer Drop: 30.0 inches
Hammer Efficiency (ER): 82.0 % (If not known, assume ER = 60% for conventiaonal drop hammer using a rope and cathead and ER = 80% for automatic trip hammer.)

Field Data												Laboratory Test Results										Total Stress (Undrained)		Effective Stress (Drained)	
Sample Number	Sample D_o (inches)	Sample D_i (inches)	Depth of Sample (ft)	Depth to Bottom of Layer (ft)	Elevation of Sample (ft)	Soil Type	N (blows/ft)	N_{60} (blows/ft)	Pocket Pen. PP (tsf)	Torvane TV (tsf)	Vane Shear VS (tsf)	Dry Unit Weight (pcf)	Moisture Content Wn (%)	Total Unit Weight (pcf)	Average Mean Grain Size D_{50} (mm)	Percent Fines (-200)	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Liquidity Index LI	qu Unconfined Compressive Strength (tsf)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)
1	3.0	2.4	2.5	4.0	384.5	CL	10	9	1.00			114.0	10.0	125.0											
2	3.0	2.4	5.0	6.5	382.0	CL	35	31	4.50			114.0	10.0	125.0											
3	3.0	2.4	7.5	9.0	379.5	SC	89	50	4.50			118.0	10.0	130.0											
4	3.0	2.4	10.0	11.0	377.0	CL-ML	91	51	4.50			122.8	11.7	137.0											
5	3.0	2.4	12.5	14.0	374.5	Rock	90	50				118.0	10.0	130.0											
6	3.0	2.4	15.0	16.0	372.0	Rock	100	56				125.9	7.3	135.0											
7	3.0	2.4	20.0	21.5	367.0	CL	54	48	4.50			122.1	11.2	136.0			32	20	12	-0.73					
8	3.0	2.4	23.0	24.0	364.0	SM	83	47				117.1	8.8	127.0											
9	3.0	2.4	25.0	26.5	362.0	CL	88	79	4.50			114.0	10.0	125.0											
10	3.0	2.4	27.5	29.0	359.5	SM	72	40				118.0	10.0	130.0											
11	3.0	2.4	30.0	31.5	357.0	SM	33	18				118.0	10.0	130.0											
12	3.0	2.4	35.0	36.5	352.0	SM	36	20				122.7	12.8	138.0											
13	3.0	2.4	40.0	41.0	347.0	SM	100	56				118.0	10.0	130.0											
14	3.0	2.4	45.0	46.0	342.0	SM	93	52				118.0	10.0	130.0											
15	3.0	2.4	50.0	51.5	337.0	SM	67	38				125.0	15.4	144.0											
16	3.0	2.4	55.0	56.5	332.0	SM	84	47				118.0	10.0	130.0											
17	3.0	2.4	60.0	61.0	327.0	SM	100	89				118.0	10.0	130.0											

NOTES

Soil Type: Enter Unified Soil Classification System Symbol.
Silt (ML) may or may not be cohesive. This worksheet models ML as a non-cohesive soil. For ML that exhibits and/or is expected to behave as a cohesive soil enter CL-ML, ML-CL, ML/CL or CL/ML for the Soil Type.

NP = Non Plastic — SPT = Standard Penetration Test
NA = Not Applicable — Liquidity Index (LI) = (Wn-PL)/(LL-PL)

Shear Wave Velocity (Vs)

A-18-003
Retaining Wall 1
Robinson Creek Bridge at Lambert Lane

Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations", Appendix A, pgs 15-18, November 2012.

Project: **Robinson Creek Bridge at Lambert Lane**
Job No: **16-278.1**
Date: **5/4/18**
Boring: **A-18-003**
Support: **Retaining Wall 1**

Hammer Efficiency (ER): 82.0 %

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Type	Undrained Shear Strength S_u (psf)	Rock *	N (bpf)	N_{SPT} (bpf) **	N_{60}	σ'_v (ksf)	σ'_v (kPa)	d Layer Thickness in upper 30 m (m)	Granular Soil Vs SAND (m/sec)	Granular Soil Vs SILT (m/sec)	Cohesive Soil Vs Cohesive[1] (m/sec)	Cohesive Soil Vs Cohesive[2] (m/sec)	Sedimentary Rock Vs (m/sec)	Profile Vs (m/sec)	Profile D/Vs (sec)
1	2.5	4.0	4.0	2.4	CL	1000		10	7	9	0.31	14.96	1.22			144	139		144	0.008
2	5.0	6.5	2.5	2.4	CL	4500		35	23	31	0.63	29.93	0.76			287	206		287	0.003
3	7.5	9.0	2.5	2.4	SC	4500		89	36	50	0.94	45.13	0.76	204					204	0.004
4	10.0	11.0	2.0	2.4	CL-ML	4500		91	37	51	1.27	61.02	0.61	220		287			220	0.003
5	12.5	14.0	3.0	2.4	Rock			90	37	50	1.61	76.92	0.91	232					232	0.004
6	15.0	16.0	2.0	2.4	Rock			100	41	56	1.94	92.72	0.61	245					245	0.002
7	20.0	21.5	5.5	2.4	CL	4500		54	35	48	2.62	125.23	1.68			287	287		287	0.006
8	23.0	24.0	2.5	2.4	SM			83	34	47	3.01	144.12	0.76	267					267	0.003
9	25.0	26.5	2.5	2.4	CL	4500		88	57	79	3.26	156.18	0.76			287	310		287	0.003
10	27.5	29.0	2.5	2.4	SM			72	30	40	3.52	168.40	0.76	273					273	0.003
11	30.0	31.5	2.5	2.4	SM			33	14	18	3.69	176.49	0.76	256					256	0.003
12	35.0	36.5	5.0	2.4	SM			36	15	20	4.05	194.01	1.52	264					264	0.006
13	40.0	41.0	4.5	2.4	SM			100	41	56	4.40	210.77	1.37	297					297	0.005
14	45.0	46.0	5.0	2.4	SM			93	38	52	4.74	226.96	1.52	300					300	0.005
15	50.0	51.5	5.5	2.4	SM			67	27	38	5.13	245.82	1.68	297					297	0.006
16	55.0	56.5	5.0	2.4	SM			84	34	47	5.49	263.01	1.52	308					308	0.005
17	60.0	61.0	4.5	2.4	SM			100	65	89	5.83	279.19	1.37	332					332	0.004
												Sum =	18.59						265	0.071

* Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
** Corrected for sample diameter

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

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

d (m)	18.59
Vs(d) (m/sec)	260
V_{S30} (m/sec)	**305**
V_{S30} (ft/sec)	**1001**

Soil Profile Type
'D' (180 m/s < Vs < 360 m/s)

Shear Wave Velocity Correlations

Sand: $Vs = \exp[4.045 + 0.096(\ln(N_{60})) + 0.236(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec
Silt: $Vs = \exp[3.738 + 0.178(\ln(N_{60})) + 0.231(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec
Cohesive[1]: $Vs = 203(S_u/P_a)^{0.475} \leq 310$ m/sec, valid for range of 100 to 310 m/sec
Cohesive[2]: $Vs = \exp[3.996 + 0.230(\ln(N_{60})) + 0.164(\ln(\sigma'_v))]$, valid for range of 100 to 310 m/sec & $N_{60} \geq 3$
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \leq 560$/m/sec
P_a = Atmospheric Pressure = 2116.2 psf

Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) for SAND and SILT based on SPT correlations are valid where $N_{60} \leq 100$ and $\sigma'_v \leq 506$ kPa.
4) Undrained Shear Strength (S_u) based on 0.5(Pocket Penetrometer) in psf.

Boring Data

R-18-001
Abutment 2
Robinson Creek Bridge at Lambert Lane

Project: **Robinson Creek Bridge at Lambert Lane**
Job No.: **16-278.1**
Date: **9/14/21**
Boring: **R-18-001**
Support: **Abutment 2**
By: **WEN**

Boring Elevation: 389.2 feet
Groundwater Depth: 29.2 feet   Groundwater Elevation: 360.0 feet
Hammer Weight: 140.0 pounds
Hammer Drop: 30.0 inches
Hammer Efficiency (ER): 82.0 % (If not known, assume ER = 60% for conventiaonal drop hammer using a rope and cathead and ER = 80% for automatic trip hammer.)

Sampler Size Conversions to SPT N-value

When sampler sizes vary from that of a SPT sampler (per ASTM 1586), conversions are needed to modify the field SPT N-values. For conversions to SPT sampler from other sampler sizes where weight (140 lb.) and height (18 inches) do not vary from ASTM 1586, the following table provides conversion values for common alternate sampler sizes.

Table 1. Conversion Values for Sampler Sizes

Sampler Type	Outside Diameter (inches)	Inside Diameter (inches)	Conversion to SPT N-value	
			Clay	Sand
1.41-inch	2.0	1.375	1.00	1.00
2-inch	2.5	1.95	0.85	0.63
2.4-inch	3.0	2.4	0.65	0.41
2.5-inch	3.875	2.5	0.24	0.13

Field Data

Sample Number	Sample D_o (inches)	Sample D_i (inches)	Depth of Sample (ft)	Depth to Bottom of Layer (ft)	Elevation of Sample (ft)	Soil Type	N (blows/ft)	N_{60} (blows/ft)	Pocket Pen. PP (tsf)	Torvane TV (tsf)	Vane Shear VS (tsf)
1	3.000	2.400	2.5	4.0	386.7	SC	100	56			
2	3.000	2.400	5.0	8.0	384.2	SC	53	30			
3	3.000	2.400	10.0	13.0	379.2	CL	42	37	4.50		
4	3.000	2.400	15.0	18.0	374.2	SM	100	56	4.50		
5	3.000	2.400	20.0	20.5	369.2	ML	100	56			
6	2.000	1.400	21.0	25.0	368.2	SM	56	77			
7	3.000	2.400	25.0	27.0	364.2	SM	100	56			
8	2.000	1.400	26.5	28.5	362.7	GM	100	137			
9	3.000	2.400	30.0	33.5	359.2	CL	61	54	2.25		
10	3.000	2.400	35.0	38.0	354.2	SM	53	30			
11	3.000	2.400	40.0	43.0	349.2	SM	100	56			
12	3.000	2.400	45.0	48.0	344.2	SM	40	22			
13	3.000	2.400	50.0	53.0	339.2	SM	100	56			
14	3.000	2.400	55.0	58.0	334.2	SM	100	56			
15	3.000	2.400	60.0	63.0	329.2	SM	100	56			
16	3.000	2.400	65.0	68.0	324.2	SM	100	56			
17	3.000	2.400	70.0	73.0	319.2	SM	100	89			
18	3.000	2.400	75.0	78.0	314.2	SM	100	89			
19	3.000	2.400	80.0	83.0	309.2	SM	100	89			
20	3.000	2.400	85.0	88.0	304.2	SM	100	89			
21	3.000	2.400	90.0	91.0	299.2	SM	100	89			

Laboratory Test Results

Sample Number	Dry Unit Weight (pcf)	Moisture Content Wn (%)	Total Unit Weight (pcf)	Average Mean Grain Size D_{50} (mm)	Percent Fines (-200)	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Liquidity Index LI	qu Unconfined Compressive Strength (tsf)	Total Stress (Undrained) Cohesion (psf)	Total Stress (Undrained) Friction Angle (degrees)	Effective Stress (Drained) Cohesion (psf)	Effective Stress (Drained) Friction Angle (degrees)
1	95.6	8.7	104.0											
2	95.6	8.7	104.0											
3	95.7	8.3	104.0											
4	117.2	9.7	129.0											
5	117.2	9.7	129.0		64									
6	117.2	9.7	129.0							0.21				
7	117.2	9.7	129.0											
8	128.8	9.7	141.0											
9	125.0	0.0	125.0											
10	111.7	18.9	133.0											
11	111.7	18.9	133.0											
12	111.7	18.9	133.0											
13	111.7	18.9	133.0											
14	111.7	18.9	133.0											
15	111.7	18.9	133.0											
16	111.7	18.9	133.0											
17	111.7	18.9	133.0											
18	111.7	18.9	133.0											
19	111.7	18.9	133.0											
20	111.7	18.9	133.0											
21	111.7	18.9	133.0											

NOTES

Soil Type: Enter Unified Soil Classification System Symbol.
Silt (ML) may or may not be cohesive. This worksheet models ML as a non-cohesive soil. For ML that exhibits and/or is expected to behave as a cohesive soil enter CL-ML, ML-CL, ML/CL or CL/ML for the Soil Type.
NP = Non Plastic   SPT = Standard Penetration Test
NA = Not Applicable   Liquidity Index (LI) = (Wn-PL)/(LL-PL)

Shear Wave Velocity (Vs)

R-18-001

Abutment 2

Robinson Creek Bridge at Lambert Lane

Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations", Appendix A, pgs 15-18, November 2012.

Project: **Robinson Creek Bridge at Lambert Lane**
Job No: **16-278.1**
Date: **9/14/21**
Boring: **R-18-001**
Support: **Abutment 2**

Hammer Efficiency (ER): 82.0 %

Sample Number	Depth (feet)	Depth to Bottom of Layer (feet)	Layer Thickness (feet)	Sample D_i (inches)	Soil Type	Undrained Shear Strength S_u (psf)	Rock *	N (bpf)	N_{SPT} (bpf) **	N_{60}	σ'_v (ksf)	σ'_v (kPa)	d Layer Thickness in upper 30 m (m)	Granular Soil Vs SAND (m/sec)	Granular Soil Vs SILT (m/sec)	Cohesive Soil Vs Cohesive[1] (m/sec)	Cohesive Soil Vs Cohesive[2] (m/sec)	Sedimentary Rock Vs (m/sec)	Profile Vs (m/sec)	Profile D/Vs (sec)
1	2.5	4.0	4.0	2.4	SC			100	41	56	0.26	12.45	1.22	152					152	0.008
2	5.0	8.0	4.0	2.4	SC			53	22	30	0.52	24.90	1.22	169					169	0.007
3	10.0	13.0	5.0	2.4	CL	4500		42	27	37	1.04	49.80	1.52			287	233		287	0.005
4	15.0	18.0	5.0	2.4	SM	4500		100	41	56	1.61	77.09	1.52	234					234	0.007
5	20.0	20.5	2.5	2.4	ML			100	41	56	2.26	107.97	0.76		265				265	0.003
6	21.0	25.0	4.5	1.4	SM			56	56	77	2.38	114.15	1.37	265					265	0.005
7	25.0	27.0	2.0	2.4	SM			100	41	56	2.90	138.85	0.61	269					269	0.002
8	26.5	28.5	1.5	1.4	GM			100	100	137	3.09	147.83	0.46	298					298	0.002
9	30.0	33.5	5.0	2.4	CL	2250		61	40	54	3.51	167.92	1.52			209	309		209	0.007
10	35.0	38.0	4.5	2.4	SM			53	22	30	3.83	183.48	1.37	271					271	0.005
11	40.0	43.0	5.0	2.4	SM			100	41	56	4.19	200.38	1.52	294					294	0.005
12	45.0	48.0	5.0	2.4	SM			40	16	22	4.54	217.28	1.52	274					274	0.006
13	50.0	53.0	5.0	2.4	SM			100	41	56	4.89	234.18	1.52	305					305	0.005
14	55.0	58.0	5.0	2.4	SM			100	41	56	5.24	251.09	1.52	310					310	0.005
15	60.0	63.0	5.0	2.4	SM			100	41	56	5.60	267.99	1.52	314					314	0.005
16	65.0	68.0	5.0	2.4	SM			100	41	56	5.95	284.89	1.52	319					319	0.005
17	70.0	73.0	5.0	2.4	SM			100	65	89	6.30	301.79	1.52	338					338	0.005
18	75.0	78.0	5.0	2.4	SM			100	65	89	6.66	318.69	1.52	342					342	0.004
19	80.0	83.0	5.0	2.4	SM			100	65	89	7.01	335.59	1.52	347					347	0.004
20	85.0	88.0	5.0	2.4	SM			100	65	89	7.36	352.50	1.52	351					351	0.004
21	90.0	91.0	3.0	2.4	SM			100	65	89	7.72	369.40	0.91	355					355	0.003
												Sum =	**27.74**						284	0.102

* Enter "rock" for Tertiary Age (<70 million years) Sedimentary Rocks. Alternatively, their "Tertiary Sand/Clay" correlation may be used.
** Corrected for sample diameter

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

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

d (m)	27.74
Vs(d) (m/sec)	272
V_{S30} (m/sec)	**282**
V_{S30} (ft/sec)	**924**

Soil Profile Type

'D' (180 m/s < Vs < 360 m/s)

Shear Wave Velocity Correlations

Sand: $Vs = \exp[4.045 + 0.096(\ln(N_{60})) + 0.236(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec

Silt: $Vs = \exp[3.738 + 0.178(\ln(N_{60})) + 0.231(\ln(\sigma'_v))]$, valid for range of 100 to 380 m/sec

Cohesive[1]: $Vs = 203(S_u/P_a)^{0.475} \leq 310$ m/sec, valid for range of 100 to 310 m/sec

Cohesive[2]: $Vs = \exp[3.996 + 0.230(\ln(N_{60})) + 0.164(\ln(\sigma'_v))]$, valid for range of 100 to 310 m/sec & $N_{60} \geq 3$

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

P_a = Atmospheric Pressure = 2116.2 psf

Notes: 1) The calculated Vs value assumes that no significant changes in the subsurface will occur to the extrapolated depth of 100 feet.
2) In the absence of in-situ measurements, limit Vs_{30} to 760 m/sec for competent rock in California.
3) The shear wave velocity (Vs) for SAND and SILT based on SPT correlations are valid where $N_{60} \leq 100$ and $\sigma'_v \leq 506$ kPa.
4) Undrained Shear Strength (S_u) based on 0.5(Pocket Penetrometer) in psf.

DRILLED SHAFT SIDE RESISTANCE AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with California Amendments

INPUT	CALCULATION

Project:	Robinson Creek Bridge at Lambert Lane (9 Piles, 2 Rows)
County / City:	Mendocino County
Job Number:	16-278.1
Boring:	R-18-002
Support Location(s):	Abutment 1

By:	KKL
Date:	1/19/22
Checked By:	WEN
Date:	1/21/22

The **Regression Coefficient (C)** for Side Resistance in Rock under normal conditions is equal to 1.0. Rock outside the normal range includes Clay Shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the normal range. ***Leave C input blank for fractured rock.***

0.75 **IGM Reduction Factor for Slurry Installation**
(Use 1.0 for Dry Hole and 0.75 for Slurry Hole)

Redundancy Factor - Use a factor of 0.8 (20% Reduction) for Non-Redundant (Single CIDH Pile) Foundations.

PILE DATA

CIDH Pile Cut-off Elevation (feet) =	374.5
CIDH Pile Diameter (inches) =	30
Socket Diameter (inches) =	
Elevation at Top of Socket (feet) =	
$f'c$ = concrete compressive strength (ksi) =	4.0
Elevation to Bottom of Permanent Casing (feet) =	
Side Resistance Reduction Factor for Steel Casing =	

Elevation at Top of Rock Socket must equal a layer break.
Elevation to Bottom of Permanent Casing must equal a layer break.

SUBSURFACE DATA

Groundwater Elevation (feet) =	368.0
Groundwater Depth below *Cut-off Elevation* (feet) =	6.5
Bottom Elev. of Lowest Liquefiable Layer = **Downdrag** (feet) =	
Overburden Stress at top of Cut-off Elevation: =	0.0
Existing Ground Surface Elevation (feet) =	
Groundwater Depth below *Existing Ground Surface* (feet) =	

Bottom Elevation of Lowest Liquefiable Layer must equal a layer break.

FOUNDATION DATA

LRFD Limit State	Geotechnical Factor	Redundancy Factor	Factored Nominal Resistance (kips)	Required Nominal Resistance (kips)	Scour Elevation* (feet)
Service	1.0	1.0	150	150	367.4
Strength	0.7	1.0	280	400	369.7
Extreme	1.0	1.0	NA	NA	372.0

**** Scour calculated per CA Amendments to AASHTO LRFD BDS 8th Ed.***

SCOUR DATA*

Long Term Scour Elevation (Degradation + Contraction) feet	Short Term Scour Depth (Local) feet
372.0	4.6

**** If no scour, then enter NA.***
Scour Elevation must equal a layer break to plot accurately on the output graph.

The analysis to determine the nominal axial resistance after the scour event considers the reduction of the effective overburden stress due to scour.

PILE GROUP EFFECTS

Soil Type	Reduction Factor for Group Effects
Cohesive	NA
Cohesionless	0.69
Rock	NA

Pile Spacing
Long Bridge = 6.4 ft
Trans Bridge = 7.7 ft

PILE GROUP CONSIDERATIONS

Pile center to center Spacing* (feet)	Pile center to center Spacing (D)	Number of Rows*
6.4	2.56	2

* Enter NA for Single Pile / CIDH Pile center to center spacing ≥ 2.5D for design

1) Caltrans 8th Ed. Amendments reverts to AASHTO 6th Ed. Equations for Noncohesive Soils / Valid Material Types: SAND, GRAVEL, CLAY, IGM and ROCK
Note: The IGM classification designates a Noncohesive (Granular) IGM.
For layers considered liquefiable enter 'YES' and enter the residual undrained shear strength for the corresponding layer.

Soil/Rock Layer	Pile Diameter (inches)	Pile Circ. (feet)	Elevation at Bottom of Layer Below Cut-off Elevation (feet)	Depth to Bottom of Layer Below Cut-off Elevation (feet)	Layer Thickness Below Cut-off Elevation (feet)	Material Type (1)	Consider Layer Liquefiable (YES)	Consider as Downdrag	Permanent Casing	Soil Total Unit Weight (pcf)	NON-COHESIVE SOIL: N_{60} (≤100) (bpf)	COHESIVE SOIL: Undrained Shear Strength Su (psf)	COHESIVE SOIL: Liquefiable Soil Layer Residual Shear Strength Sr (psf)	ROCK: Rock Quality Designation (RQD)	ROCK: Uniaxial Compressive Strength q_u (psi)	ROCK: Uniaxial Compressive Strength q_u (ksf)	Sound Rock, Little Fractured: Regression Coefficient C	Sound Rock, Little Fractured: Drilled Shaft Concrete Compressive Strength $f'c$ (ksf)	Jointed / Fractured Rock: Joint Modification Factor α_E Closed Joints CALCULATED	Jointed / Fractured Rock: Joint Modification Factor α_E Open Joints CALCULATED	Jointed / Fractured Rock: Selected for Analysis	NOTE: IF "ERROR", READ CELL AC3 FOR REGRESSION COEFFICIENT
1	30	7.85	373.2	1.3	1.3	SAND	NO			125	55											
1	30	7.85	371.5	3.0	1.7	SAND	NO			125	55											
1	30	7.85	369.7	4.8	1.8	SAND	NO			125	55											
1	30	7.85	367.4	7.1	2.3	SAND	NO			125	55											
1	30	7.85	365.5	9.0	1.9	SAND	NO			125	55											
2	30	7.85	340.5	34.0	25.0	SAND	NO			130	36											
3	30	7.85	333.0	41.5	7.5	SAND	NO			135	58											
3	30	7.85	332.0	42.5	1.0	SAND	NO			135	58											
3	30	7.85	310.5	64.0	21.5	SAND	NO			135	58											
4	30	7.85	278.0	96.5	32.5	SAND	NO			135	70											

Drilled Shafts: Construction Procedures and Design Methods, FHWA-IF-99-025, 1999
Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016, FHWA GEC 010, May 2011

DRILLED SHAFT SIDE RESISTANCE
AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with California Amendments

INPUT	CALCULATION

The **Regression Coefficient (C)** for Side Resistance in Rock under normal conditions is equal to 1.0. Rock outside the normal range includes Clay Shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the normal range. ***Leave C input blank for fractured rock.***

Project:	Robinson Creek Bridge at Lambert Lane (9 Piles, 2 Rows)
County / City:	Mendocino County
Job Number:	16-278.1
Boring:	R-18-001
Support Location(s):	Abutment 2

By:	KKL
Date:	1/19/22
Checked By:	WEN
Date:	1/21/22

0.75 — IGM Reduction Factor for Slurry Installation (Use 1.0 for Dry Hole and 0.75 for Slurry Hole)

Redundancy Factor - Use a factor of 0.8 (20% Reduction) for Non-Redundant (Single CIDH Pile) Foundations.

PILE DATA

CIDH Pile Cut-off Elevation (feet) =	374.5
CIDH Pile Diameter (inches) =	30
Socket Diameter (inches) =	
Elevation at Top of Socket (feet) =	
f'c = concrete compressive strength (ksi) =	4.0
Elevation to Bottom of Permanent Casing (feet) =	
Side Resistance Reduction Factor for Steel Casing =	

Elevation at Top of Rock Socket must equal a layer break.
Elevation to Bottom of Permanent Casing must equal a layer break.

SUBSURFACE DATA

Groundwater Elevation (feet) =	368.0
Groundwater Depth below *Cut-off Elevation* (feet) =	6.5
Bottom Elev. of Lowest Liquefiable Layer = **Downdrag** (feet) =	
Overburden Stress at top of Cut-off Elevation: =	0.0
Existing Ground Surface Elevation (feet) =	
Groundwater Depth below *Existing Ground Surface* (feet) =	

Bottom Elevation of Lowest Liquefiable Layer must equal a layer break.

FOUNDATION DATA

LRFD Limit State	Geotechnical Factor	Redundancy Factor	Factored Nominal Resistance (kips)	Required Nominal Resistance (kips)	Scour Elevation* (feet)
Service	1.0	1.0	150	150	369.8
Strength	0.7	1.0	280	400	371.5
Extreme	1.0	1.0	NA	NA	373.2

*** Scour calculated per CA Amendments to AASHTO LRFD BDS 8th Ed.***

SCOUR DATA*

Long Term Scour Elevation (Degradation + Contraction) feet	Short Term Scour Depth (Local) feet
373.2	3.4

** If no scour, then enter NA.*
Scour Elevation must equal a layer break to plot accurately on the output graph.

The analysis to determine the nominal axial resistance after the scour event considers the reduction of the effective overburden stress due to scour.

PILE GROUP EFFECTS

Soil Type	Reduction Factor for Group Effects
Cohesive	NA
Cohesionless	0.69
Rock	NA

Pile Spacing
Long Bridge = 6.4 ft
Trans Bridge = 7.7 ft

PILE GROUP CONSIDERATIONS

Pile center to center Spacing* (feet)	Pile center to center Spacing (D)	Number of Rows*
6.4	2.56	2

* Enter NA for Single Pile / CIDH Pile center to center spacing ≥ 2.5D for design

1) Caltrans 8th Ed. Amendments reverts to AASHTO 6th Ed. Equations for Noncohesive Soils / Valid Material Types: SAND, GRAVEL, CLAY, IGM and ROCK
Note: The IGM classification designates a Noncohesive (Granular) IGM.

For layers considered liquefiable enter 'YES' and enter the residual undrained shear strength for the corresponding layer.

Soil/Rock Layer	Pile Diameter (inches)	Pile Circ. (feet)	Elevation at Bottom of Layer Below Cut-off Elevation (feet)	Depth to Bottom of Layer Below Cut-off Elevation (feet)	Layer Thickness Below Cut-off Elevation (feet)	Material Type (1)	Consider Layer Liquefiable (YES)	Consider as Downdrag	Permanent Casing	Soil Total Unit Weight (pcf)	NON-COHESIVE SOIL: N_{60} (≤100) (bpf)	COHESIVE SOIL: Undrained Shear Strength Su (psf)	COHESIVE SOIL: Liquefiable Soil Layer Residual Shear Strength Sr (psf)	ROCK: Rock Quality Designation (RQD)	ROCK: Uniaxial Compressive Strength q_u (psi)	ROCK: Uniaxial Compressive Strength q_u (ksf)	ROCK – Sound Rock, Little Fractured: Regression Coefficient C	ROCK – Sound Rock, Little Fractured: Drilled Shaft Concrete Compressive Strength *f'c* (ksf)	ROCK – Jointed / Fractured Rock: Joint Modification Factor α_E – Closed Joints CALCULATED	ROCK – Jointed / Fractured Rock: Joint Modification Factor α_E – Open Joints CALCULATED	ROCK – Jointed / Fractured Rock: Selected for Analysis	NOTE: IF "ERROR", READ CELL AC3 FOR REGRESSION COEFFICIENT
1	30	7.85	373.2	1.3	1.3	SAND	NO			125	60											
1	30	7.85	371.5	3.0	1.7	SAND	NO			125	60											
1	30	7.85	369.8	4.7	1.7	SAND	NO			125	60											
1	30	7.85	369.5	5.0	0.3	SAND	NO			125	60											
1	30	7.85	368.0	6.5	1.5	SAND	NO			125	60											
1	30	7.85	361.0	13.5	7.0	SAND	NO			125	60											
2	30	7.85	360.0	14.5	1.0	CLAY	NO			125	54	2200										
2	30	7.85	356.0	18.5	4.0	CLAY	NO			125	54	2200										
3	30	7.85	343.5	31.0	12.5	SAND	NO			133	36											
3	30	7.85	342.0	32.5	1.5	SAND	NO			133	36											
3	30	7.85	341.0	33.5	1.0	SAND	NO			133	36											
4	30	7.85	340.0	34.5	1.0	SAND	NO			135	70											
4	30	7.85	339.0	35.5	1.0	SAND	NO			135	70											
4	30	7.85	338.0	36.5	1.0	SAND	NO			135	70											
4	30	7.85	337.0	37.5	1.0	SAND	NO			135	70											
4	30	7.85	324.0	50.5	13.0	SAND	NO			135	70											
4	30	7.85	298.0	76.5	26.0	SAND	NO			135	70											

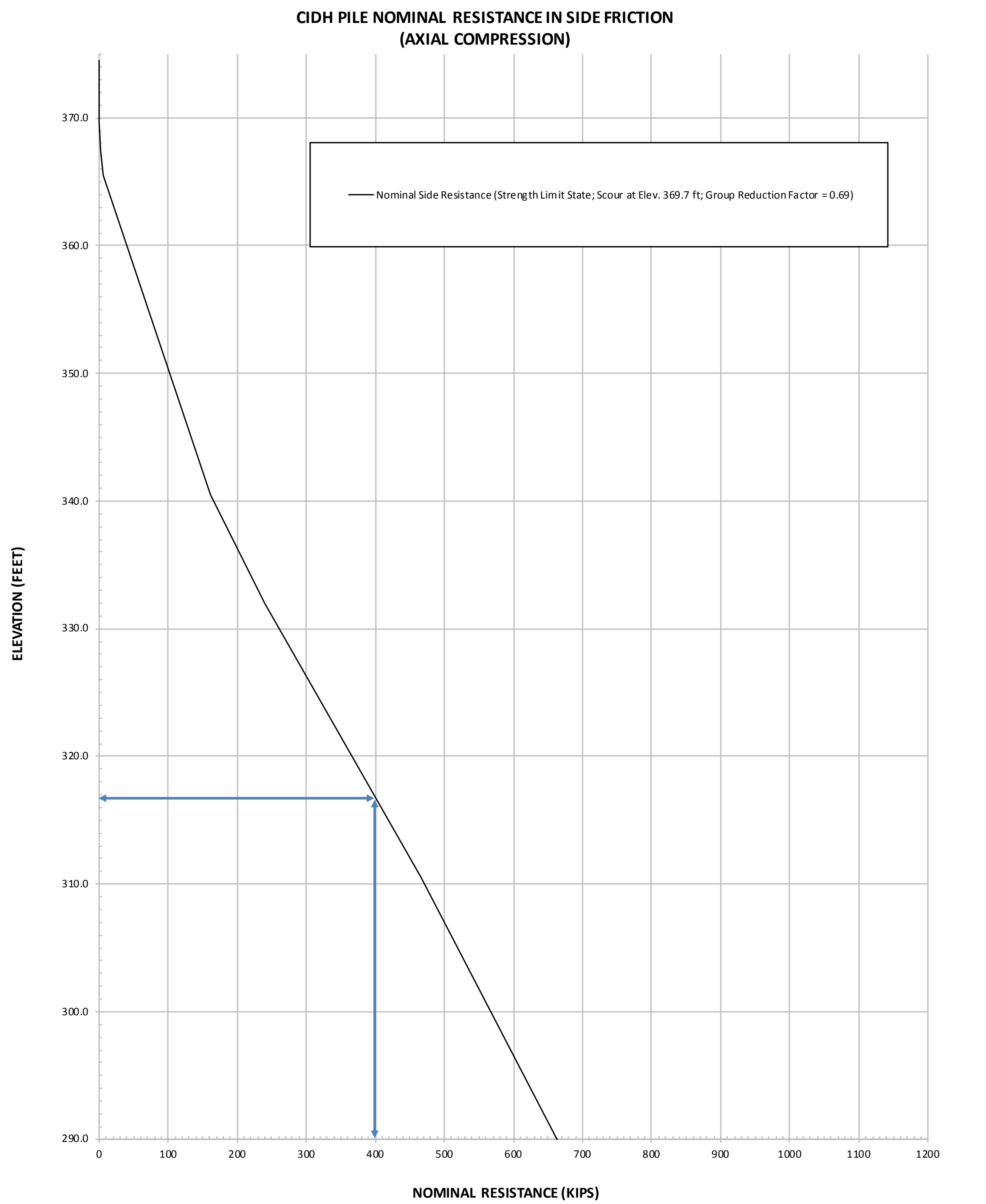
Drilled Shafts: Construction Procedures and Design Methods, FHWA-IF-99-025, 1999
Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016, FHWA GEC 010, May 2011

CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE IN SIDE FRICTION
Robinson Creek Bridge at Lambert Lane (9 Piles, 2 Rows)
Mendocino County
Crawford Project Number: 16-278.1

Crawford & Associates, Inc.
Geotechnical Engineering, Design and Construction Services
January 19, 2022

Support Location(s): Abutment 1	**Pile Diameter = 30 inches**	**Pile Cut-Off Elevation = 374.5 feet**
Boring(s): R-18-002	**Socket Diameter = NA**	**Permanent Casing Tip Elevation = NA**

SERVICE LIMIT			**STRENGTH LIMIT**			**EXTREME LIMIT**		
REQUIRED NOMINAL RESISTANCE =	150	kips	REQUIRED NOMINAL RESISTANCE =	400	kips	REQUIRED NOMINAL RESISTANCE =	NA	kips
SCOUR ELEVATION =	367.4	feet	SCOUR ELEVATION =	369.7	feet	SCOUR ELEVATION =	372.0	feet
DESIGN PILE TIP ELEVATION =	NA	feet	DESIGN PILE TIP ELEVATION =	314.0	feet	DESIGN PILE TIP ELEVATION =	NA	feet



CIDH Pile Nominal Side Resistance calculated consistent with 2017 8th Edition AASHTO LRFD Bridge Design Specifications with California Amendments. **No End Bearing Contribution.**

CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE IN SIDE FRICTION

Robinson Creek Bridge at Lambert Lane (9 Piles, 2 Rows)

Mendocino County

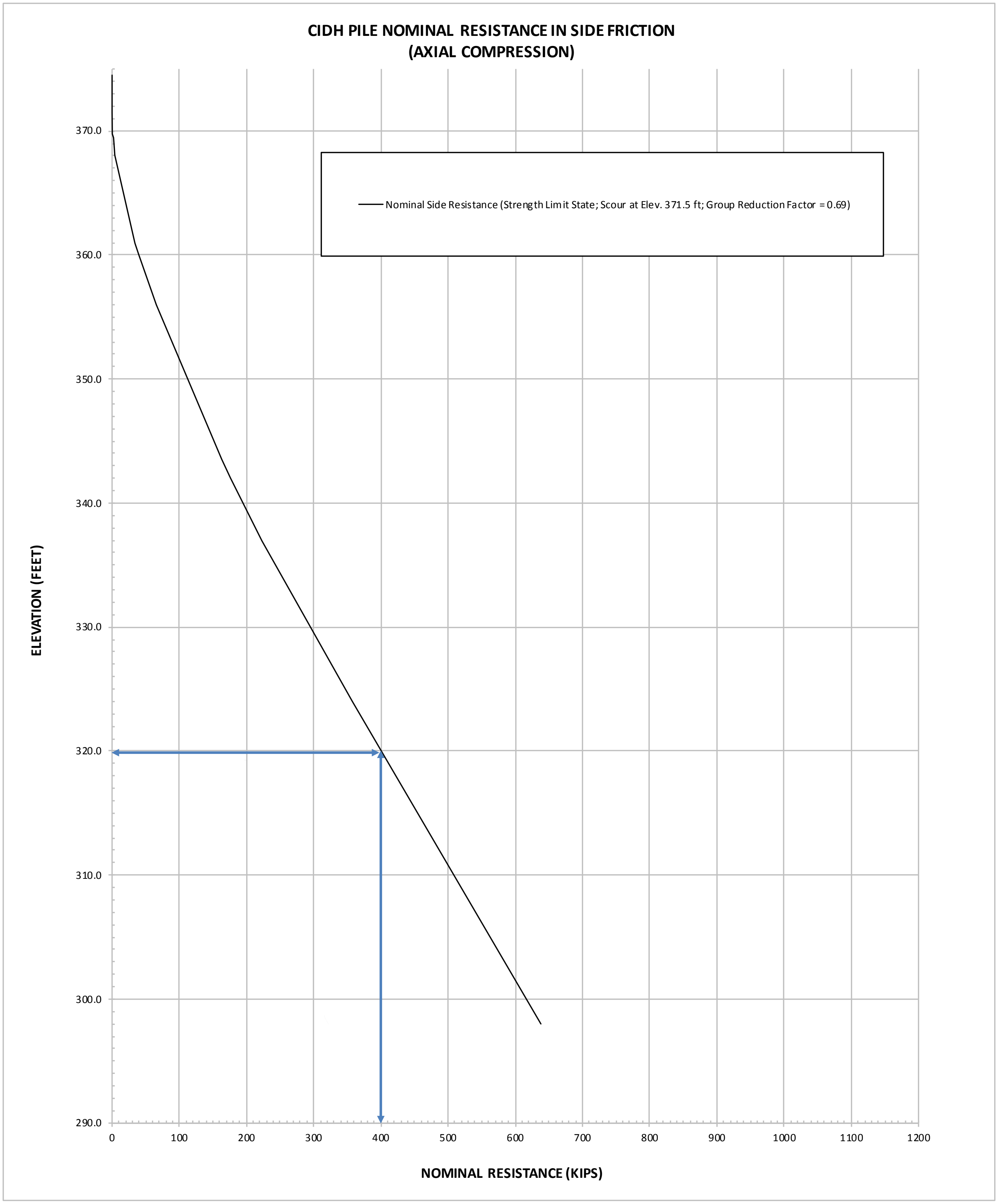
Crawford Project Number: 16-278.1

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

January 19, 2022

Support Location(s): Abutment 2	**Pile Diameter = 30 inches**	**Pile Cut-Off Elevation = 374.5 feet**
Boring(s): R-18-001	**Socket Diameter = NA**	**Permanent Casing Tip Elevation = NA**

SERVICE LIMIT			STRENGTH LIMIT			EXTREME LIMIT		
REQUIRED NOMINAL RESISTANCE =	150	kips	REQUIRED NOMINAL RESISTANCE =	400	kips	REQUIRED NOMINAL RESISTANCE =	NA	kips
SCOUR ELEVATION =	369.8	feet	SCOUR ELEVATION =	371.5	feet	SCOUR ELEVATION =	373.2	feet
DESIGN PILE TIP ELEVATION =	NA	feet	DESIGN PILE TIP ELEVATION =	314.0	feet	DESIGN PILE TIP ELEVATION =	NA	feet



CIDH Pile Nominal Side Resistance calculated consistent with 2017 8th Edition AASHTO LRFD Bridge Design Specifications with California Amendments. **No End Bearing Contribution.**

NOMINAL BEARING RESISTANCE -- SPREAD FOOTING FOUNDATION

AASHTO LRFD Bridge Design Specifications, Eighth Edition withCalifornia Amendments April 2019

Date:	1/19/22	*Support:*	Retaining Wall 1
Project:	Replacement of Robinson Creek Bridge at Lambert Lane	*Boring:*	R-18-003
Project No:	16-278.1	*Base of Footing:*	382.8 ft

Basic Formulation (Munfakh, et al, 2001)

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma BN_{\gamma m} C_{w\gamma}$$ **(Equation 10.6.3.1.2a-1)**

in which:

$N_{cm} = N_c s_c i_c$ (Equation 10.6.3.1.2a-2)

$N_{qm} = N_q s_q d_q i$ (Equation 10.6.3.1.2a-3)

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (Equation 10.6.3.1.2a-4)

Bearing Capacity Factors from Table 10.6.3.1.2a-1

N_c = 42.16

N_q = 29.44

N_γ = 41.06

where:

q_n = gross nominal bearing resistance

c = cohesion (psf)

B' = effective footing width (feet)

γ = total (moist) unit weight of soil (pcf)

D_f = footing embedment depth (feet)

N_c, N_q, and N_γ = bearing capacity factors

C_{wq} & C_{wg} = correction factors for location of groundwater

s_c, s_γ, and s_q = footing shape correction factors

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

i_c, i_γ, and i_q = load inclination factors

Input Parameters

γ =	120.0	(pcf)
ϕ =	34.0	(degrees)
c =	0.0	(psf)
D_f =	0.0	(feet) below lowest adjacent finished grade
D_w =	0.0	(feet) below lowest adjacent finished grade

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

Table 10.6.3.1.2a-2

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

D_w = depth of groundwater below lowest adjacent finished grade (feet)

Solve for Nominal Bearing Resistance

Case	Footing Dimensions B'	L'/B'	L'	1.5B+D_f	C_{wq}	$C_{w\gamma}$	s_c	s_γ	s_q	Gross Nominal Bearing Resistance	Factored Gross Nominal Bearing Resistance (ksf) — Resistance Factor (φ_b): Service	Strength	Extreme
	(feet)			(feet)						(ksf)	1.0	0.55	0.80
Srv I	6.70	4.2	28.00	10.05	0.500	0.500	1.17	0.90	1.16	7.46	7.46	--	--
Strgth I	5.20	5.4	28.00	7.8	0.500	0.500	1.00	1.00	1.00	6.41	--	3.52	--
Ext I	4.80	5.8	28.00	7.2	0.500	0.500	1.00	1.00	1.00	5.91	--	--	4.73
Ext II	2.70	10.4	28.00	4.05	0.500	0.500	1.00	1.00	1.00	3.33	--	--	2.66
1													
2													
3													
4													
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5													
6													

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

Shape Correction Factors

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

Schmertmann's Modified Method for Calculation of Immediate Settlement [Schmertmann et al. (1978)]

Reference: Soils and Foundations - Volume II, FHWA NHI-06-089, pgs 8-45 to 8-53, December 2006.

$$S_i = C_1 C_2 \Delta p \sum \Delta H_i \quad \text{where} \quad \Delta H_i = H_c\,(I_z/XE)$$

where: S_i = Immediate Settlement (inches).

C_1 = correction factor to incorporate the effect of strain relief due to embedment = $1 - 0.5(Po/\Delta P) \geq 0.5$

C_2 = correction factor to incorporate time-dependent increase in settlement for t (years) after construction = $1 + 0.2\log_{10}[10t(\text{years})]$

P_o = **effective** overburden pressure at foundation level (psf).

ΔP = **net** uniform applied stress at the foundation level (psf).

P_{op} = **effective** overburden pressure (psf) at depth Izp below footing.

H_i = soil layer within the depth of influence (inches) below footing.

H_c = layer thickness (inches) below footing.

t = time to settlement (years). [t = 0.1 year for end of construction]

P = gross uniform bearing stress (psf)

E = elastic modulus of soil layer (tsf).

X = factor used to determine the value of the elastic modulus.
X = 1.25 for axisymmetric case (L'/B' = 1)
X = 1.75 for plane strain case (L'/B' $\geq$ 10)

I_z = strain influence factor.

I_{zb} = strain influence factor at the base of the footing.
I_{zb} = 0.1 for axisymmetric case (L'/B' = 1)
I_{zb} = 0.2 for plane strain case (L'/B' $\geq$ 10)

I_{zp} = maxium strain influence factor = $0.5 + 0.1(\Delta P/P_{op})^{0.5}$

D_I = maximum depth of influence below footing (feet)
D_I = 2B' for axisymmetric case (L'/B' = 1)
D_I = 4B' for plane strain case (L'/B' $\geq$ 10)

Effective Footing Width, B' =	6.70	feet
L'/B' =	4.18	
Effective Footing Length, L' =	28.00	feet
Depth to Bottom of Footing =	0.0	feet
Groundwater Depth Below Ground Surface =	0.0	feet
Vertical Load =		kips
ΔP =	1000	psf
t (years) =	1.2	

I_{ZB} =	0.135	
Izp =	0.69	
D_{IP} =	4.53	feet
D_I =	18.13	feet
P =		psf
Po =	0.0	psf
Pop =	291.3	psf
C_1 =	1.000	
C_2 =	1.2	
ΔP =	1000.0	psf
$\sum \Delta H_i$ =	0.2117	in/tsf

NOTES

ENTERING A VALUE FOR ΔP OVER RIDES THE VERTICAL LOAD INPUT VALUE.

ΔP IS EQUAL TO THE NET UNIFORM BEARING STRESS.

IMMEDIATE SETTLEMENT, S_i = 0.129 inches

Soil Layer	SOIL LAYERS BELOW GROUND SURFACE: Depth to Bottom of Layer Below Ground Surface (feet)	Layer Thickness (feet)	Unified Soil Classification	Total Soil Unit Weight (pcf)	$N1_{60}$	Soil Type (1, 2, 3 or 4)	Elastic Modulus (E) (tsf)	X
1	1.5	1.5	CL	125	29	1	116	1.43
2	3.0	1.5	SM	130	30	4	360	1.43
3	7.3	4.3	CL	125	43	1	172	1.43
4	13.3	6.0	SP	130	52	4	624	1.43
5	16.3	3.0	CL	125	48	1	192	1.43
6	23.3	7.0	CL	62	48	1	192	1.43
7	33.3	10.0	SM	67	26	2	182	1.43

SOIL LAYERS BELOW FOOTING: Bottom of Layer (feet)	Layer Thickness (feet)
1.5	1.5
3.0	1.5
7.3	4.3
13.3	6.0
16.3	3.0
23.3	7.0
33.3	10.0

SOIL LAYERS ABOVE D_{IP}: Layer Thickness Hc (feet)	Depth to Layer Mid-Point Z_1 (feet)	I_z at Z_i	XE (tsf)	ΔHi (in/tsf)
1.5	0.8	0.227	166	0.0246
1.5	2.3	0.411	515	0.0144
1.5	2.3	0.413	246	0.0309
			(Soil Layers Above D_{IP}) $\sum \Delta H_i$ =	0.06982

SOIL LAYERS BELOW D_{IP}: Layer Thickness Hc (feet)	Depth to Layer Mid-Point Z_1 (feet)	I_z at Z_i	XE (tsf)	ΔHi (in/tsf)
2.8	5.9	0.619	246	0.0844
6.0	10.3	0.396	892	0.0319
3.0	14.8	0.167	275	0.0219
1.8	17.2	0.046	275	0.0036
			(Soil Layers Below D_{IP}) $\sum \Delta H_i$ =	0.14188

Total $\sum \Delta H_i$ = 0.2117

SOIL TYPE	E
1) Silts, sandy silts, slightly cohesive mixtures	$4N1_{60}$
2) Clean fine to medium sands and slightly silty sands	$7N1_{60}$
3) Coarse sands and sands with little gravel	$10N1_{60}$
4) Sandy gravel and gravels	$12N1_{60}$

PROJECT: Robinson Creek Bridge at Lambert Lane
PROJECT NUMBER: 16-278.1
SUPPORT LOCATION(S): Retaining Wall 1
BORING NUMBER(S): R-18-003
BY: KKL **CHECK:** WEN
DATE: 1/19/22 **DATE:** 1/21/22

APPENDIX VI

Caltrans Comments and Crawford & Associates, Inc. Response

Office of Special Funded Projects
Comment & Response Form

(Revised 08/2011)

General Project Information (OSFP Liaison to complete)		**Review Phase** (OSFP Liaison to complete)	**Reviewer Information** (Reviewer Liaison to complete)	
Dist:	**04**	☐ PSR/PDS (Review No.)	Reviewer Name:	**Justin Anderson, Chris McMahon**
Proj ID (Phase):	**0000020504-N**	☐ APS/PSR (Review No.)	Functional Unit:	Office of Geotechnical Design – West
Project Name:	**Lambert Lane Bridge over Robinson Creek**	☐ APS/PR (Review No. 1)	Cost Center:	3660/59
OSFP Liaison:	**Ling Wang**	☑ Type Selection	Phone Number:	**(510) 414-9122**
Phone:	**(916) 227-8483**	☐ 65% PS&E Unchecked Details	e-mail:	**Jusitn.anderson@dot.ca.gov**
E-mail:		☐ PS&E (Review No.1)	Date of Review:	**06/30/22**
		☐ Construction	Structure Name*:	**Lambert Lane Bridge over Robinson Creek**
		☐ Other:	Br No*:	10C-0146
			(*Use if necessary to when comment sheets are by individual structure)	

Consultant Information (to be filled in by Consultant)

Consultant Geotechnical Lead (First and Last Name)	Geotechnical Consultant Firm	Phone Number	E-mail	Response Date
W. Eric Nichols	**Crawford & Associates, Inc.**	**916-717-9906**	**eric.nichols@crawford-inc.com**	**07/06/2022**

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
1	FR	6, groundwater	Section does not mention groundwater for R-18-003. Please add	The groundwater in R-18-003 was/is shown in Table 3 of the Foundation Report. The text in this section is revised for clarity.	
2	FR	6, groundwater	Please state what the design groundwater elevation is	Groundwater used in geotechnical analysis was/is included under the Geotechnical Design Parameters Section in Appendix V. Section 6 in the report has been revised to also include the groundwater elevation used in geotechnical analysis.	

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

✓= Comment Resolved (for Reviewer's use)

Submittal Data (Reviewer to complete)
Project ID: 0000002050 Reviewer Str Name*: Prefumo Cr Bike Br
Date of Review: Functional Unit: Br No*. 49-0261Z *=if applicable

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓
3	FR	7, Table 4	I could not locate this information in the provided hydraulic report	Scour data for this project was provided by Wreco in their Bridge Design Hydraulic Study Report dated August 5, 2021, Executive Summary, Scour Data Table, page viii and also Section 5.6, Table 10, page 37.	
4	FR	8, corrosion	Current corrosion guidelines set the minimum resistivity to not test for chlorides or sulfates at 1,500 ohm-cm	This section has been revised to show 1,500 ohm-cm.	
5	FR	8, Table 6	Please include VS30 calculations for confirmation of values	VS30 calculations are now included in Appendix V.	
6	FR	11, recommendations	Foundation type recommendations should not be included in the Final Foundation Report since the foundation type has already been selected	Section 11 has been revised.	
7	FR	12, Table 8	Please fix the table headers	Headers have been fixed.	
8	FR	13, Table 9,10	Were settlement design tips left out on purpose?	Yes. The piles will be founded in very dense materials. Refer to Note 5, Table 9 and Note 4, Table 10 in Section 13. Also refer to the Settlement Section under Deep Foundations (Bridge Abutments) in Appendix V for further explanation.	
9	FR	Page 15 Table 13	Factored Bearing Resistance should be ksf, not psf	Table 13 is revised to show ksf.	
10					
11					
12					
13					

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OSFP Rev Form 12/2010

Submittal Data (Reviewer to complete)
Project ID: 0000002050 Reviewer Str Name*: Prefumo Cr Bike Br
Date of Review: Functional Unit: Br No*. 49-0261Z *=if applicable

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	✓

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✓= Comment Resolved
(for Reviewer's use)

OSFP Rev Form 12/2010