FINAL FOUNDATION REPORT

NCRCD Sulphur Creek Fish Passage (Project #30144)

St. Helena, California
Crawford File No. 20-643.1

Prepared by:

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August 23, 2024

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August 23, 2024
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Mr. Jon Sampson
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FINAL FOUNDATION REPORT
NCRCD-Sulphur Creek Fish Passage (Project #30144)
St. Helena, California

Dear Mr. Sampson,

Crawford & Associates, Inc. (Crawford) prepared this **FINAL** Foundation Report for the NCRCD-Sulphur Creek Fish Passage (Project #30144) located in Napa County, California. We prepared this report in accordance with our agreement dated July 9, 2020, and Amendment 1 dated January 6, 2023, between Crawford and Mark Thomas. This report supersedes our draft Foundation Report, dated September 5, 2023.

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.,

Ellen Tiedemann

Ellen Tiedemann, PE
Senior Engineer

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1 INTRODUCTION

1.1 PURPOSE

Crawford & Associates, Inc. (Crawford) prepared this **FINAL** Foundation Report for the NCRCD-Sulphur Creek Fish Passage Project in St. Helena, California in accordance with our agreement dated July 9, 2020, and Amendment 1 dated January 6, 2023, between Crawford and Mark Thomas. The purpose of this report is to provide earth materials criteria for use in the design of the proposed new bridge foundations. It includes the results of the subsurface exploration, laboratory testing results, the Log of Test Borings (LOTB), and foundation recommendations for the proposed bridge.

We understand that project design is in accordance with Caltrans procedures, guidelines, standards and specifications that use Load and Resistance Factor Design (LRFD) method for bridge design.

1.2 GEOTECHNICAL SERVICES

To prepare this report, Crawford:

- Discussed the project goals and objectives with Jon Sampson from Mark Thomas and Andrew Smith from WRA;
- Performed three seismic refraction lines and surface geologic reconnaissance of the site and immediate vicinity at the site on November 10, 2020;
- Drilled and sampled two exploratory borings to a maximum depth of 28.4 feet (ft) below ground surface (bgs) on January 5, 2021;
- Published a draft Geotechnical Memorandum on March 1, 2021;
- Published a draft Preliminary Foundation Memorandum on January 10, 2022;
- Completed site reconnaissance on September 1, 2020, and January 11, 2023;
- Drilled and sampled two exploratory borings on January 26-27, 2023, at the proposed abutments to depths of 60 to 62.1 ft bgs;
- Reviewed the 60% Site Plan sheet by WRA dated January 19, 2023;
- Reviewed the 65% plans provided by Mark Thomas on April 12, 2023;
- Reviewed loading provided by Mark Thomas on July 25, 2024;
- Reviewed the 100% General Plan provided by Mark Thomas on August 16, 2024;
- Reviewed available published topographic, geologic and seismic mapping of the site vicinity;
- Completed laboratory testing on soil and rock samples obtained during the subsurface exploration, and;
- Performed engineering evaluations and analyses to develop the recommendations contained in this report.

This final Foundation Report supersedes the draft Foundation Report dated September 5, 2023. Limitations of this study are discussed in the final section of this report.

2 PROJECT DESCRIPTION

The project site is located on the western city limits of St. Helena, about 1.8 miles west of State Route 29, where a private road crosses over Sulphur Creek. The site is approximately at latitude 38.4879°N and longitude 122.4816°W. The project location and vicinity are shown Figure 1 in Appendix I.

The Napa County Resource Conservation District (NCRCD) proposes to remove an existing fish ladder (originally installed in 2002) within Sulphur Creek channel east of the existing bridge. To allow for more enhanced fish passage, the design team determined that the new channel bed at the bridge should be lowered about three to four feet below existing channel grade. However, the existing bridge is considered scour critical and will be replaced to accommodate the planned channel grading.

The existing bridge, built in the early 1900s, is about a 28 ft long and 12 ft wide single lane, single span reinforced concrete structure. The bridge is scour critical with the spread foundations exposed within the channel. At both abutments, repairs have been attempted to protect against scour effects.

The design team previously evaluated utilizing retaining walls to protect the existing bridge foundations during the channel regrading (refer to Crawford's Draft Geotechnical Memorandum dated March 1, 2021). The project now includes a new bridge located upstream (west) of the existing bridge.

Based on conversations with Mark Thomas, we understand the bridge to be a 55-foot long by 22-foot wide single-span, prefabricated steel bridge. The substructure is shown as seat-type wall abutments with cantilever wingwalls at each abutment. Both abutments are shown supported by 30-inch diameter, cast-in-drilled-hole (CIDH) piles. The new deck grade is shown at elev. 315.01 ft at Abutment 1 (Begin Bridge "SC" Line Sta. 20+34.72) and elev. 319.14 ft at Abutment 2 (End Bridge "SC" Line Sta. 20+89.72). The cutoff elevation at Abutment 1 and 2 is 307.5 ft and 311.5 ft, respectively. The bridge crosses over Sulphur Creek perpendicularly along its alignment.

The existing bridge will be removed as part of this project. Channel regrading is anticipated upstream and downstream of the proposed bridge.

All elevations in this report are based on the NAVD 1988 vertical datum.

3 GEOTECHNICAL INVESTIGATION

Field investigation for replacement of the existing bridge at Sulphur Creek consisted of three seismic survey lines and four exploratory borings. Table 1 provides a subsurface investigation summary.

Table 1: Subsurface Investigation Summary

Seismic Survey

Line Number/Type	Completion Date	Seismometer Type	Length (ft)
S-1/Refraction	11/10/2021	24 channel ES-3000	100±
S-2/Refraction	11/10/2021	24 channel ES-3000	100±
S-3/Refraction	11/10/2021	24 channel ES-3000	100±

Exploratory Borings

Boring Number	Completion Date	Drill Rig Type	Hammer Efficiency[1] (%)	Approx. Ground Surface Elevation (ft)	Boring Depth (ft)
A-21-001	1/5/2021	CME 55 Truck	89.3	316.3	28.4
A-21-002	1/5/2021	CME 55 Truck	89.3	318.9	20.3
R-23-003	1/16/2023	CME 55 Track	81.1	315	60.0
R-23-004	1/17/2023	CME 55 Track	81.1	319	62.1

[1] A hammer energy calibration test was not performed specifically for this project/site. The hammer efficiency shown was reported by the driller at the time of the field exploration. Hammer type was automatic for both drill rigs.

3.1 SEISMIC REFRACTION SURVEY

A seismic refraction survey was completed by Crawford on November 10, 2020. The seismic survey consisted of three seismic refraction surveys (S-1, S-2, and S-3) to determine the approximate depth to rock and evaluate rippability characteristics along the proposed channel regrading alignment. The seismic lines were about 100 ft long and were completed within the channel upstream and at the existing bridge. The locations of seismic refraction lines are shown on Figure 2A.

The data was recorded with a 24 channel ES-3000 seismometer with geophones arranged in a line running generally east to west for S-1 and southwest to northeast for S-2 and S-3. Twenty-one geophones were used for S-1, S-2, and S-3. The energy source for this testing was a 40-lb falling weight with an approximate 24-inch drop striking a steel plate at various locations along the geophone spread. The recorded data was analyzed using the Geometrics, Inc. SeisImager/SW software package. Refraction seismic profiling indicates primary wave (compression wave) velocities. The refraction profiles and locations are shown in Figures 5A/B and Figure 2A, respectively. Reynicole Gilbert and Amando Castro were the field personnel for this field study.

3.2 GEOTECHNICAL BORINGS

3.2.1 GEOTECHNICAL DRILLING

Crawford retained Geo-Ex Subsurface Exploration (Geo-Ex) in 2021 to drill and sample two borings (A-21-001 and A-21-002) at the existing bridge. The borings were located along the private road on either side of the bridge. Geo-Ex utilized a CME 55 truck-mounted drill rig to complete the borings with 4-inch diameter solid-stem auger and 3.8-inch diameter side discharging mud rotary.

Crawford also retained Geo-Ex in 2023 to drill and sample two borings (R-23-003 and R-23-004) at the proposed bridge abutments. Geo-Ex utilized a CME 55 track-mounted drill rig to complete the borings with 4-inch diameter solid-stem auger, 3.8-inch diameter side discharging mud rotary, and rock core drilling equipment.

In boring A-21-001, caving occurred at about 20 ft below the ground surface, therefore, 3.8-inch diameter mud rotary drilling was utilized for the rest of the boring. Auger refusal (characterized as near maximum drill rig effort) was encountered in borings A-21-001 and A-21-002 at elev. approximately 288.3 and 298.9 ft, respectively.

Auger refusal (characterized as near maximum drill rig effort) was encountered in borings R-23-003 and R-23-004 at approximately elev. 266 ft and 258 ft, respectively. Moderate water loss circulation was encountered in the rock core run for borings R-23-004 between approximately elev. 304 and 308 ft.

3.2.2 SAMPLING PROTOCOL

Soil and decomposed to moderately weathered rock samples were recovered by means of 2.0-inch O.D. Standard Penetration Test (SPT) split-spoon sampler (ASTM D1586) and a 3.0-inch O.D. “Modified 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. Some samples of rock were recovered with HQ size diamond core barrels and retained in core boxes.

Drive samples taken in the borings were typically collected at approximate 5 ft intervals and as otherwise directed by the field engineer. 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 and identified on the logs as 50/x, where x is the depth of penetration in inches.

Selected portions of recovered soil/rock samples 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.

3.2.3 LOGGING

Crawford’s field personnel logged the exploratory borings with the Unified Soil Classification System (USCS) and the Caltrans Logging Manual[2]. The borings were logged and earth materials field-classified by a geologist as to consistency, color, texture and gradation on the bases of penetration resistance, examination of samples and observation of drill cuttings. Where diamond coring was used to advance the borings, the recovered cores were logged as percent recovery,

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[2] Caltrans, Soil and Rock Logging, Classification, and Presentation Manual, 2010 Edition with Errata Sheet (August 2018).

Rock Quality Designation, grain size, degree of weathering, hardness and fracture density. Kennedy Hauder was the field geologist for this study.

3.2.4 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 guidance by Caltrans[3], 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 and borings logs provided in Appendix III and N_{60} values are shown for borings R-23-003 and R-23-004 in Appendix V.

3.2.5 BOREHOLE ABANDONMENT

At completion, the exploratory borings were backfilled with cement grout in accordance with the county boring permit requirements.

3.2.6 BORING LOCATIONS AND ELEVATIONS

The boring locations were measured in the field with respect to existing site features and then referenced to project stationing. The boring elevations are referenced to project datum and were estimated based on site topography provided by WRA and 65% Plans provided by Mark Thomas. The locations and details of exploratory borings are shown on Figure 2B (Appendix I) and the LOTB drawing and boring logs (Appendix III).

4 LABORATORY TESTING PROGRAM

Crawford completed laboratory tests on selected representative samples obtained from the exploratory borings to aid in soil/rock 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, lateral spreading, deep foundations, and corrosion potential.

The following laboratory tests have been completed on representative soil and rock samples obtained from the exploratory borings include:

- Corrosivity Testing (CTM 643, CTM 417, and CTM 422)
- Gradation (ASTM D6913)
- Moisture Content/Unit Weight (ASTM D2216/D7263)

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[3] Caltrans, Geotechnical Manual, Sampler Size Conversions to SPT N-value, Soil Correlations Module (March 2021).

- R-value (CTM 301)
- Unconfined Compressive Strength (ASTM D2166)

Laboratory summary and test results are provided in Appendix IV and the exploratory boring locations are shown on Figure 2B in Appendix I and the LOTB included in Appendix III.

5 GEOTECHNICAL CONDITIONS

5.1 SITE GEOLOGY

5.1.1 REGIONAL GEOLOGY

The project is located in the Coast Ranges Geomorphic Province[4] of California. The Coast Ranges are northwest-trending mountain ranges (with typical mountain peaks at 2,000 to 4,000 ft and occasionally 6,000 ft elevation above sea level) and valleys. The Coast Ranges are composed of thick Mesozoic and Cenozoic sedimentary strata that have a complex structure due to intense folding and faulting. The northern and southern ranges are separated by a depression containing the San Francisco Bay.

The northern Coast Ranges are dominated by irregular, knobby, landslide-prone material of the Franciscan Complex. In places, the Franciscan rocks are overlain by volcanic cones and flows of the Quien Sabe, Sonoma, and Clear Lake volcanic fields. The eastern border is characterized by strike-ridges and valleys in Upper Mesozoic strata that dip beneath alluvium of the Great Valley that extends to the east. To the west is the Pacific Ocean. The coastline is uplifted, terraced and wave-cut.

The Coast Ranges are subparallel to the active San Andreas Fault. The San Andreas is more than 600 miles long, extending from Point Arena to the Gulf of California. West of the San Andreas is the Salinian Block, a granitic core extending from the southern extremity of the Coast Ranges to the north of the Farallon Islands.

5.1.2 LOCAL GEOLOGY

At the bridge site, published geologic mapping[5] of the area shows Sulphur Creek underlain by Holocene aged (11,000 years) modern steam channel deposits (Qhc) consisting of loose alluvial sand, gravel, and silt within active, natural channels. Geologic mapping also shows White Sulphur Springs Rd at/upstream of the bridge underlain by Holocene aged stream terrace point bar and overbank deposits (Qht), consisting of sand, gravel, silt, and clay. Adjacent to the southern abutment and along the southern bank upstream of the bridge, the site is shown to be underlain by Jurrassic-Cretaceous aged Franciscan graywacke (KJfs) which consists of thickly bedded graywacke with minor interbedded shale. Franciscan Complex mélange (KJfm), a tectonic mixture of sandstone, greenstone, chert, garbbo, and metamorphic rocks imbedded in a sheared shaley matrix, is mapped about 200 ft northeast of the site.

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[4] California Geologic Survey (2002), *California Geomorphic Province*, Note 36.

[5] Clahan, K.B., Wagner, D.L., Bezore, S.P., Sowers, J.M., and Witter, R.C., 2005, Geologic map of the Rutherford 7.5-minute quadrangle, Sonoma and Napa counties, CA: A Digital database, v.1.0, California Geological Survey, series unknown, 1:24,000.

Landslide deposits are mapped approximately 1,850 ft south of the site. During our November 2020 field investigation, Crawford observed a local bank landslide about 50 ft long and 30 to 40 ft tall. During the November 2020 site visit, we observed burnt trees and vegetation caused by the 2020 Glass Fire Complex. Based on our experience, the loss of vegetation is expected to cause local bank destabilization to the existing over-steepened slopes. Based on observations during the field investigations, the proposed bridge abutments appear to be outside of the mapped landslides.

No other evidence of significant geologic hazards (such as faulting, volcanoes, settlement, very soft soils, springs, subsidence, etc.) was observed at the project site as part of this study. The bridge site is not in a tsunami inundation zone. A geologic map of the site is included as Figure 3 in Appendix I.

5.2 SURFACE CONDITIONS

The bridge site is located in a generally rural area and land use near the bridge site is privately owned, undeveloped land. The nearest structure to the site is a private residence located about 500 ft east of the proposed bridge.

On the north side of the proposed bridge is a soil shoulder which parallels White Sulphur Springs Road. South of Sulphur Creek at the proposed southern abutment is an unpaved, narrow, private access road. Directly south of the access road are grassy slopes with abundantly scattered trees and grass.

In the vicinity of the bridge, Sulphur Creek flows generally east/northeast at the proposed bridge location. Sulphur Creek constricts to about 15 ft wide as it flows easterly under the existing bridge. Based on conversations with the land owner, the channel geometry has meandered over time. The land owner observed the channel water course change after the 2014 earthquake in Napa.

Within Sulphur Creek, large coarse materials (up to cobbles/boulders) line the bottom of the channel and its slopes. Rock outcrop was present along the southern slope, southwest of the existing bridge. The northern bank was heavily vegetated while the southern bank was over-steepened due channel erosion. Along the eastern side of the existing northern abutment and on either side of the southern abutment, heavy rock has been placed to protect the banks from scour. At the northern abutment, a concrete wall approximately 21 ft long runs along the western bank.

The channel was dry during our September 2020 field review and had less than 6-inches of water (under the bridge) during our November 2020, January 2021, and 2023 field explorations, and approximately one foot of water during our 2023 site visit. The channel bottom (thalweg) at the existing bridge is at about elev. 304.3 ft, about 12.5 ft below the existing bridge deck.

An overhead utility line follows generally along the south side of White Sulphur Springs Road and then crosses north over White Sulphur Springs Road approximately 250 ft west of the existing bridge. No underground utilities were identified by USA North 811 members. Locations of other utilities, if present, are unknown.

Observations made at the site during the site visits are generally consistent with the referenced mapping. Coarse granular soils (sand/gravel/cobbles and boulders) are present within the channel and along the banks of the creek. Local outcrops of sedimentary rock were present up/downstream of the existing bridge within and along the banks of the channel.

5.3 SUBSURFACE CONDITIONS

Based on the exploratory boring data, subsurface materials underlying the bridge site are considered consistent with the published mapping.

5.3.1 EXPLORATORY BORINGS

We divided materials encountered in our borings into two general soil/rock units considered significant to the proposed project. Refer to the LOTB and 2021 boring logs in Appendix III for more specific soil/rock descriptions, boring details, and elevations. Caltrans' Standard Plans[6] provide an explanation of terms and engineering geology descriptors used to log the soil and bedrock.

Unit 1 (Alluvium Deposits/Roadway Fill) consists of alluvial deposits that are generally comprised of clayey sand, clayey sand with gravel and cobbles, and poorly-graded gravel with clay and cobbles. The apparent density of the granular soil varies from dense to very dense. This unit was encountered in all the exploratory borings. The depths and elevations to which Unit 1 materials were encountered are shown in Table 2. Unit 1 materials were encountered to greater depths north of Sulphur Creek (borings A-21-001 and R-23-003) than those south of Sulphur Creek (borings A-21-002 and R-23-004).

Table 2: Depth/Elevation of Bottom of Unit 1 Soils

Boring Number	Approximate Depth to Bottom of Layer (feet)	Approximate Elevation (feet)
A-21-001	17	299
A-21-002	8	311
R-23-003	23	292
R-23-004	6	313

Unit 2 (Decomposed/Intensely Weathered Rock) was encountered below Unit 1 and generally consists of decomposed to moderately weathered, very soft to moderately soft, sedimentary rock (graywacke and shale). In all the exploratory borings, this unit was drilled with solid stem augers and/or using a tricone bit mud rotary drilling equipment, and sampled with primarily a SPT sampler. The N_{60} values range from 36 to greater than 100 blows per foot.

Borings A-21-001, A-21-002 were terminated in Unit 2 materials at solid-stem auger refusal at 28.4 feet and 20.2 feet (about elev. 288 and 299 feet), respectively. In boring R-22-003, this unit was cored between 50.5 to 60 feet bgs. Boring R-23-004 was cored between 11 to 15 feet bgs but had poor recovery and fluid loss through the interbedded layers of weathered rock and was continued with a tricone bit.

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[6] 2023 Standard Plans A10F/A10G (Legend – Soil) and A10H (Legend – Rock)

5.3.2 SEISMIC REFRACTION SURVEY

Interpreted seismic refraction survey profiles indicate primary wave (V_p) velocities ranging from about 3,000 to 15,000 feet per second (fps) for unconsolidated granular surficial soils and underlying rock. The interpreted results/details of the seismic refraction surveys are summarized in Table 3.

Table 3: Summary of Seismic Refraction Survey

Seismic Line	Approx. Depth from Thalweg to Bottom of Layer (ft)	Approx. Elevation Range at Bottom of Layer (ft)	Approximate Primary Wave Velocity, V_p (fps)
S-1	7 to 15	299 to 311	3,000 to 4,000
	--	--	4,000 to 10,500
S-2	1 to 14	288 to 309.5	3,700 to 4,000
	--	--	4,000 to 8,600
S-3	13	291	4,000
	--	--	15,000

The refraction profiles and locations are shown in Figure 6A through 6B and Figure 2a, respectively.

6 GROUNDWATER

Crawford measured groundwater at a depth of 14.2 ft (elev. 300.6) in boring R-23-003 during the 2023 exploration, and at a depth of 17.5 ft (elev. 298.8 ft) in boring A-21-001 during the 2021 exploration. Both groundwater measurements were recorded north of the existing bridge, or near Abutment 1. Groundwater was not established due to rotary drilling methods in borings south of the existing bridge, or near Abutment 2. Less than twelve inches of flowing water was present in the channel at the time of our 2021 and 2023 field explorations.

Groundwater levels can fluctuate due to changes in precipitation, creek water levels, irrigation, pumping of wells, drought and other factors. We expect that groundwater level will be coincident with the creek water level in the vicinity of the bridge since Sulphur Creek flows year-round. The groundwater elevation used for design is elev. 305 ft based on the 2021 and 2023 borings.

7 AS-BUILT FOUNDATION DATA

No as-built foundation data for this structure was available for review and the bridge foundation type is not known.

8 SCOUR DATA

Scour data provided by WRA is summarized in Table 4.

Table 4: Scour Data

Support Location	Thalweg Elev. (ft)	Long Term Scour (Degradation and Contraction) Elevation (ft)	Short Term Scour (Local) Depth (ft)
Abut 1	304.0	302.4	NA
Abut 2		302.4	NA

The proposed pile cutoff elevations for Abutment 1 and Abutment 2 are located 5.1 feet and 9.1 feet above scour elevation, respectively. Scour is therefore not expected to detrimentally impact the new bridge foundations provided that bridge design and construction adhere to the pile foundation recommendations contained in this report.

9 CORROSION EVALUATION

For structural elements, Caltrans[7] 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 1,100 ohm-cm. The results of corrosivity tests on combined weathered rock samples (from 16 to 21 ft bgs) obtained from the boring completed for this study 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 (?)
R-23-003 / 2A & 3A	16-21	298.8-293.8	7.54	2,600	1.8	7.6	No

Test results summarized above and current Caltrans guidelines indicate a “non-corrosive” soils for structural concrete/steel foundation elements. The test results are only an indicator of soil corrosivity. Section 12 the 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.

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[7] Caltrans, Corrosion Guidelines Version 3.2, May 2021

10 SEISMIC INFORMATION

10.1 SHEAR WAVE VELOCITY

Using SPT N-values corrected for hammer efficiency and the equations outlined by Caltrans[8], a correlated shear wave velocity (V_{S30}) in the upper 100±ft (30 meters) of 486 meters per second (m/s) (about 1,593 ft/sec) is considered appropriately conservative for use in new bridge design. This value corresponds to "very dense soil and soft rock" with 360 m/s < Vs < 760 m/s for the upper 100 ft of the soil profile.
The correlated V_{S30} values estimated from the 2023 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 feet	
					V_{S30} (m/sec)	V_{S30} (ft/sec)
Abutment 1	R-23-003	314.8	254.8	60.0	504	1,653
Abutment 2	R-23-004	318.9	256.8	62.1	467	1,532
				Average V_{S30} =	486	1,594

10.2 SOIL CLASSIFICATION

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

According to Caltrans[9], 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, $V_{S30} > 886$ 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 scourable soil and weathered rock.

The simplified design method as specified in Section 6.2.3.2 of SDC is not allowed for piles founded in Class S2 soil and lateral analysis as specified in Section 6.2.4.2 of SDC is required.

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[8] Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021.
[9] Seismic Design Criteria (SDC) Version 2.0

10.3 GROUND MOTION HAZARD

The Caltrans ARS Online (V3.0.2)[10] 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.

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 near-fault adjustment factor does apply to this site, whereas the basin factor does not apply.

The mean magnitude value reported by 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 RECOMMENDED SEISMIC DATA

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:

- Shear Wave Velocity, V_{S30}: 1,594 feet/second (486 meters/second);
- PGA: 0.65g;
- Magnitude (M) at PGA: 6.72; and
- Mean Site-to-Fault Distance at 1.0 Second: 8.9 mi (14.3 km).
-

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

10.4 OTHER SEISMIC HAZARDS

10.4.1 SURFACE FAULT RUPTURE

The site does not lie within an Alquist–Priolo Earthquake Fault Zone (EFZ) and no known active faults are mapped by the California Geologic Survey[11] (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. The nearest active seismic source, located about 15.9± miles southwest of the site, is the Rodgers Creek Fault Zone. An inactive fault

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[10] https://arsonline.dot.ca.gov/, accessed 11/11/2022.

[11] http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

(undifferentiated Quaternary age) of the Browns Valley Fault Zone (West Napa Fault) is mapped about 0.10± miles east 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 EVALUATION

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.

Granular soils below groundwater levels at the site had SPT blow count $(N_1)_{60} \geq 30$. Therefore, the potential for liquefaction does not exist at this site and is not a geotechnical design consideration.

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 the project design.

10.4.4 SEISMIC SLOPE STABILITY

No indications of slope instability were observed in the vicinity of proposed bridge abutments. The potential for seismic instability of the existing creek banks is considered to be low and limited to potential for minor (surficial) distortion along the natural creek banks.

10.4.5 LATERAL SPREADING POTENTIAL

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.[12] indicate that potentially liquefiable soil layers with SPT $(N_1)_{60}$ values 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.

11 GEOTECHNICAL RECOMMENDATIONS

Based on the foregoing, the site is considered stable with support available for the proposed bridge. Conditions are suitable for the installation of the planned CIDH piles at the abutments penetrating into the underlying bedrock (Unit 2). Specific recommendations for CIDH piles are provided below.

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[12] American Society of Civil Engineers (ASCE) Journal of Geotechnical and Geoenvironmental Engineering, December 2002.

Based on the geotechnical data developed for this project, CIDH piles can provide adequate axial geotechnical resistance and minimize construction noise and vibration. Such piles would achieve support within the underlying "weathered" rock through side friction and designed with assured penetration of bearing materials for consideration of long-term security with respect to scour. 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 to avoid construction delays should groundwater be present during construction. The "wet" method requires placement of inspection tubes to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) of the CIDH pile.

Spread footing foundations for the bridge abutments would need to be placed below scour depths for long-term security, and require large/deep open excavations near/in the creek channel. Due to the depth to competent and secure bearing materials and construction considerations within in the channel (excavation slopes, shoring, sediment control, etc.), the use of spread footing foundations at this site, although feasible, does not appear appropriate.

Driven piles are not recommended at the side due to potential hard driving conditions within the weathered/decomposed rock.

Geotechnical considerations included excavatability within the cobbles and decomposed/moderately weathered rock.

11.1 DEEP FOUNDATIONS

11.1.1 PILE FOUNDATION DATA AND LOADING

Foundation data and loading for the proposed pile foundations provided by Mark Thomas are presented in Table 7 and 8.

Table 7: Foundation Design Data

Support No.	Design Method	Pile Type	Finished Grade Elevation (ft)	Cutoff Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement – Service Load (in)[1]	Number of Piles per Support
					B	L		
Abut 1	LRFD	30-inch CIDH	314.7	307.9	3.5	16.0	2	3
Abut 2	LRFD	30-inch CIDH	318.7	311.9	3.5	16.0	2	3

[1] Based on Caltrans' current practice, the total permissible settlement is two inches for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Table 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)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	310	105	465	155	N/A	N/A	N/A	N/A	N/A	N/A
Abut 2	310	105	465	155	N/A	N/A	N/A	N/A	N/A	N/A

11.1.2 FOUNDATION DESIGN RECOMMENDATIONS

The CIDH pile nominal resistance was evaluated using Load and Resistance Factor Design (LRFD) methods and factors from AASHTO LRFD Bridge Design Specifications (8th Edition) with Caltrans amendments. Groundwater was modeled at elev. 305 ft. The top of the pile to depth of scour below the pile cap are excluded from contributing to geotechnical resistance.

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 summaries the results of our pile compression resistance and lateral resistance analyses.

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

Table 9: Foundation Design Recommendations

Support No.	Pile Type	Cutoff Elev. (ft)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (inches)	Nominal Resistance[3] (kips)				Design Tip Elev. (ft)	Specified Tip Elev. (ft)
						Strength/Const.		Extreme			
			Total	Perm.		Comp. $\varphi = 0.7$	Tens. $\varphi = 0.7$	Comp. $\varphi = 1.0$	Tens. $\varphi = 1.0$		
Abut 1	30" CIDH	307.9	255.79	N/A	2.0	230	N/A	N/A	N/A	270 (a) 282 (b)	270
Abut 2	30" CIDH	311.9	255.79	N/A	2.0	230	N/A	N/A	N/A	276 (a) 286 (b)	276

Notes:

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

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	230	N/A	270 (a) 282 (b)	270
Abut 2	30” CIDH	230	N/A	276 (a) 286 (b)	276

Notes:

1) Design tip elevations are controlled by: (a) Compression, (b) Lateral Load.
2) The Specified Tip Elevation should not be raised above the design tip elevation.
3) Column heading modified from Required Nominal Resistance to Nominal Resistance.
4) The piles will be embedded adequately into rock, and the piles will not be subjected to downdrag loads; therefore, a detailed assessment of the pile group settlement is not considered warranted.

11.2 APPROACH FILLS

11.2.1 EARTHWORK

Site grading and general earthwork should be performed in accordance with Section 17 and Section 19 of Caltrans Standard Specifications[13], respectively. General preparation should include stripping and disposal of all debris and organic material to at least 5 feet (laterally) outside fill limits. All materials unsuitable for use as fill should be properly disposed of off-site.

11.2.2 FILL MATERIAL

The source of borrow material for construction of embankment fills has not been identified. Any imported fill should be approved by the resident engineer prior to transporting to the site, should have 100% passing 3-inch sieve and have low expansion potential [Expansion Index (EI) < 50 and Sand Equivalent (SE) > 20]. Imported fill used at and below subgrade level should also be required to meet or exceed that of the design R-value. In general, all fill material should be free of debris and organic material.

11.2.3 FILL PLACEMENT AND COMPACTION

Construct embankment and place new fill in accordance with Caltrans Standard Specifications, including at least 95% relative compaction per CTM 216 on all fill within 150 feet of bridge abutments. Soil should be placed in thin lifts (6 to 8-inches) prior to compaction.

Where new fill is placed against an existing slope or when constructing half the embankment width at a time, prepare the slope by cutting into it at least 6 feet horizontally and below any

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[13] Caltrans 2023 Standard Specifications

loose/soft or otherwise unsuitable materials as the new embankment is placed in layers (consistent with Section 19 of Caltrans Standard Specifications).

11.2.4 SLOPE GEOMETRY AND STABILITY

Fill heights for the approach embankments are unknown, but anticipated to be less than five feet. Based on boring data generated for this study, the near-surface soils are capable of providing adequate support for shallow fill heights.

Due to natural meandering of Sulphur Creek, and the existing scour conditions at the project site, slope protection measures such as RSP placement near the bridge abutments may be considered in order to maintain stability.

11.2.5 EROSION CONTROL

Soils used for embankment construction are considered at least locally susceptible to erosion and provisions for erosion control (such as planting, erosion control mats, etc.) are recommended. Over-side runoff from pavement should be controlled by use of curbs, dikes, down-drains, gutters, etc. Local sloughing is expected to be controllable by typical maintenance procedures.

11.2.6 SETTLEMENT

The encountered upper unit materials at this site are considered capable of sustaining anticipated fill loads without significant distress and with no more than nominal settlement (on order of 1-inch or less), mostly occurring as the load is applied. No waiting period is required from end of fill placement to start of foundation installation.

11.3 LATERAL EARTH PRESSURES

The material placed behind each abutment/wingwall is expected/recommended to meet Structure Backfill requirements consistent with Caltrans Standard Specifications. The equivalent fluid weights (EFWs) shown in Table 11 are recommended to design the abutment, wing walls, (assuming fully drained and level backfill conditions).

Table 11: Recommended Equivalent Fluid Weights (EFW)

Condition	**Static**		**Incremental Seismic**
	Coefficient k (dim.)	EFW (pcf)	Δ EFWEQ (pcf)
Active	0.28	37	22

The EFW values shown above assume:

- Level backfill condition;
- Caltrans Structure Backfill with soil unit weight (γ) = 130 pcf and minimum angle of internal friction (ϕ) = 34°;
- PGA = 0.65g;
- Horizontal seismic acceleration coefficient (k_h) ≤ 0.22;
- Vertical seismic acceleration coefficient (k_v) = 0.0; and

- Drainage behind walls is placed in accordance with Caltrans Standard Plans and Specifications.

11.3.1 STATIC LATERAL EARTH PRESSURE

Caltrans allows use of static active earth pressure for embankment behind seat-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.

Assume 0.002*H of relative wall movement is required for the active condition to apply; otherwise, use the at-rest condition (to be provided upon request).

11.3.2 SEISMIC LATERAL EARTH PRESSURE

For seismic design, add the incremental lateral seismic active earth pressure to the static active earth pressure. For Structural Backfill behind the abutments, the incremental active seismic coefficient Δk_{AEQ} is taken as 0.25x(PGA) using Equation 8.4 presented by Augusti and Sitar[14]. A PGA of 0.65 was used for this site/project.

A triangular pressure distribution should be used and the magnitude of the resultant should be applied at H/3 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 0.28 times the design surcharge pressure, or 0.28-times a minimum surcharge of 240 psf.

11.4 STRUCTURAL SECTION AND ROADWAY SUBGRADE

Crawford completed one R-value test (CTM 301) on a bulk sample of anticipated subgrade soils. The test results indicate an R-value of 65 by Stabilometer. A design R-value of 50 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 12.

Table 12: New Pavement Structural Sections

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

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[14] Seismic Earth Pressures on Retaining Structures in Cohesionless Soils, 2013

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 construction should conform to Caltrans Standard Specifications unless otherwise stated in the Special Provisions. The project specifications should explicitly exclude vibration, impact or grouting installation methods if not approved by the permit documents for the project. This could consist of noise or vibration concerns, 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

Due to the anticipated presence of groundwater at the abutments, 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 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 49-1.03:

Expect moderately difficult pile drilling due to the conditions shown in the following table:

Support location	*Conditions*
Abutment 1	*Decomposed to moderately weathered, very soft to moderately hard Shale and Graywacke.*
Abutment 2	*Intensely weathered to moderately weathered, very soft to moderately soft Shale and Graywacke.*

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.

13 NOTES FOR CONSTRUCTION

13.1 DEEP FOUNDATIONS

The contractor should anticipate variable drilling conditions in all CIDH pile excavations due to the presence of dense to very dense granular soils, cobbles, and decomposed to slightly weathered rock. 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 or core barrel) to advance the drilled hole excavation within the zones of interbedded layers of harder rock as necessary. Crawford encountered drilling fluid loss while drilling the borings; the contractor should be prepared for potential seepage through the cobbles and weathered rock.

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.

If utilized, the temporary casing should be set in a drilled hole and should be removed during placement of concrete. If an oscillator or rotator is used to construct the CIDH piles, the following is recommended:

- The contractor should be prepared for subsurface soil/rock conditions that include layers of loose to very dense granular soils and decomposed to slightly weathered rock within CIDH pile foundation depths.
- The contractor should maintain a positive fluid head within the drill rod at all times. The fluid should be mineral or polymer slurry; water may be 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 consider 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 one continuous operation. The rebar cage should be suspended throughout the concrete pour.

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 EXISTING UTILITIES

An overhead utility line follows generally along the shoulder of the south side of White Sulphur Springs Road then crosses north over White Sulphur Springs Road approximately 215 ft west of the proposed bridge. A county owned utility pole is located approximately 29 feet east of the existing bridge. The presence/absence of any underground utilities should be confirmed prior to construction. Utilities should be protected during construction.

13.3 EXISTING FOUNDATIONS

New bridge foundations are proposed to be constructed upstream of the existing bridge structure. We do not expect the existing foundations to impact the construction of the proposed pile foundations for the bridge supports.

13.4 EXCAVATION AND SHORING

Based on the anticipated soil conditions, we expect excavation of the upper surficial soils can be achieved with typical heavy construction equipment at the bridge abutments.

The Caterpillar Handbook[15] estimates shale—the bedrock type we encountered—is rippable with a CAT D9R with a single ripping shank up to a primary wave (p-wave) velocity of 7,400 fps, marginally rippable up to a p-wave velocity of 8,000 fps, and non-rippable with a p-wave velocity above 9,500 fps. Based on our review of the plans, the channel will be excavated to a maximum 6 ft bgs. Our seismic results generally indicate the materials within the upper 7 to 15 ft have a p-wave velocity of 4,000 fps. Near S-2, we observed P-wave velocities of 8,400 fps within a few feet of existing grade likely indicating some harder rock may be encountered closer to the surface during construction and require additional excavation effort and or the use of pneumatic hammers and/or large equipment.

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.

13.5 DEWATERING

Soils/rock below groundwater/creek water level are considered capable of transmitting seepage to open excavations. The pile footing elevations at the abutments are above the groundwater level encountered in the borings completed for this study. However, nuisance water within foundation excavations may be present during construction and cannot be precluded. Therefore, Type D structure excavation (per Caltrans) is considered appropriate to show on the plans at those locations.

Winter or spring construction, or periods during or following rain, can expect higher water surface level in the creek and may also encounter higher/perched groundwater levels. If nuisance water is encountered within foundation excavations, the contractor should be prepared to dewater excavations with sump pumps and/or by means of diking/diversion of surface water (if present). The bottom of the abutment/pile cap excavations may be soft/wet. If needed, the use of coarse,

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[15] Caterpillar Handbook of Ripping, 12th Edition

granular soils (e.g. aggregate base or drain rock) at the base of excavation would be expected to provide an appropriate working surface.

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;
- Review submittals and requests for information pertaining to pile installation.
- Observe pile installation 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/rock conditions differ 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

Crawford performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. Do not use this report for different locations and/or projects without the written consent of Crawford. Where referenced, we used ASTM or Caltrans standards as a general (not strict) guideline only. We do not warranty our services.

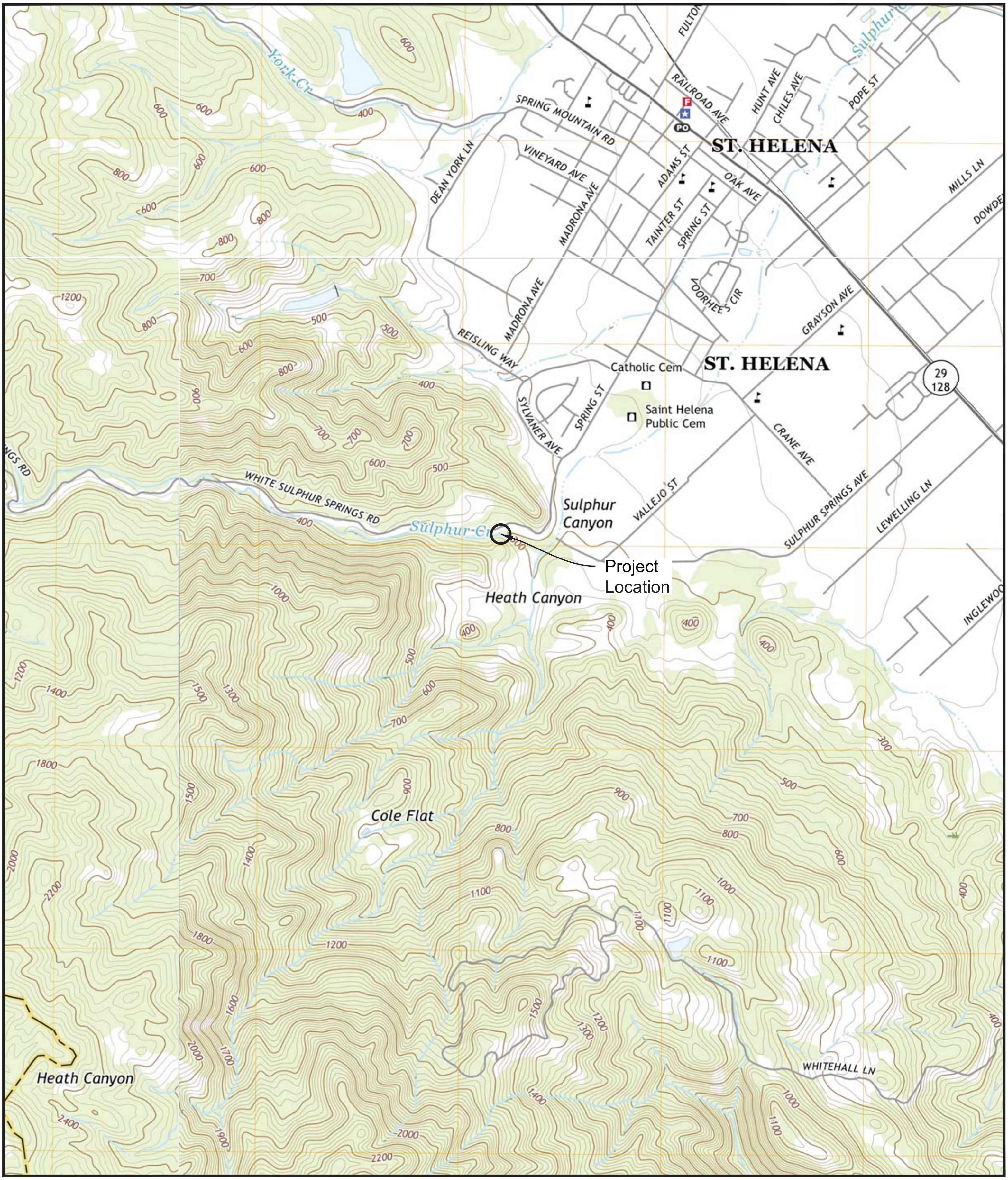
Crawford based this report on the current site conditions. We assumed the soil, rock, and groundwater conditions are representative of the subsurface conditions on the site. Actual conditions between explorations will vary along the project alignment.

Our scope did not include evaluation of flooding potential, aerial photograph review, or toxicology. Please contact Crawford if you would like an evaluation of one or more of these potentially damaging issues.

Logs and an LOTB of our explorations are presented in Appendix III. The lines designating the interface between soil types are approximate. The transition between soil types may be abrupt or gradual. Our recommendations are based on the final logs, which represent our interpretation of the field logs and general knowledge of the site and geological conditions.

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

APPENDIX I



North

Source:

USGS 7.5' Topographic Maps, Calistoga, Sonoma County, California, 2018, Scale: 1:24,000

USGS 7.5' Topographic Maps, Kenwood, Sonoma County, California, 2018, Scale: 1:24,000

USGS 7.5' Topographic Maps, Rutherford, Sonoma County, California, 2018, Scale: 1:24,000

USGS 7.5' Topographic Maps, Saint Helena, Sonoma County, California, 2018, Scale: 1:24,000

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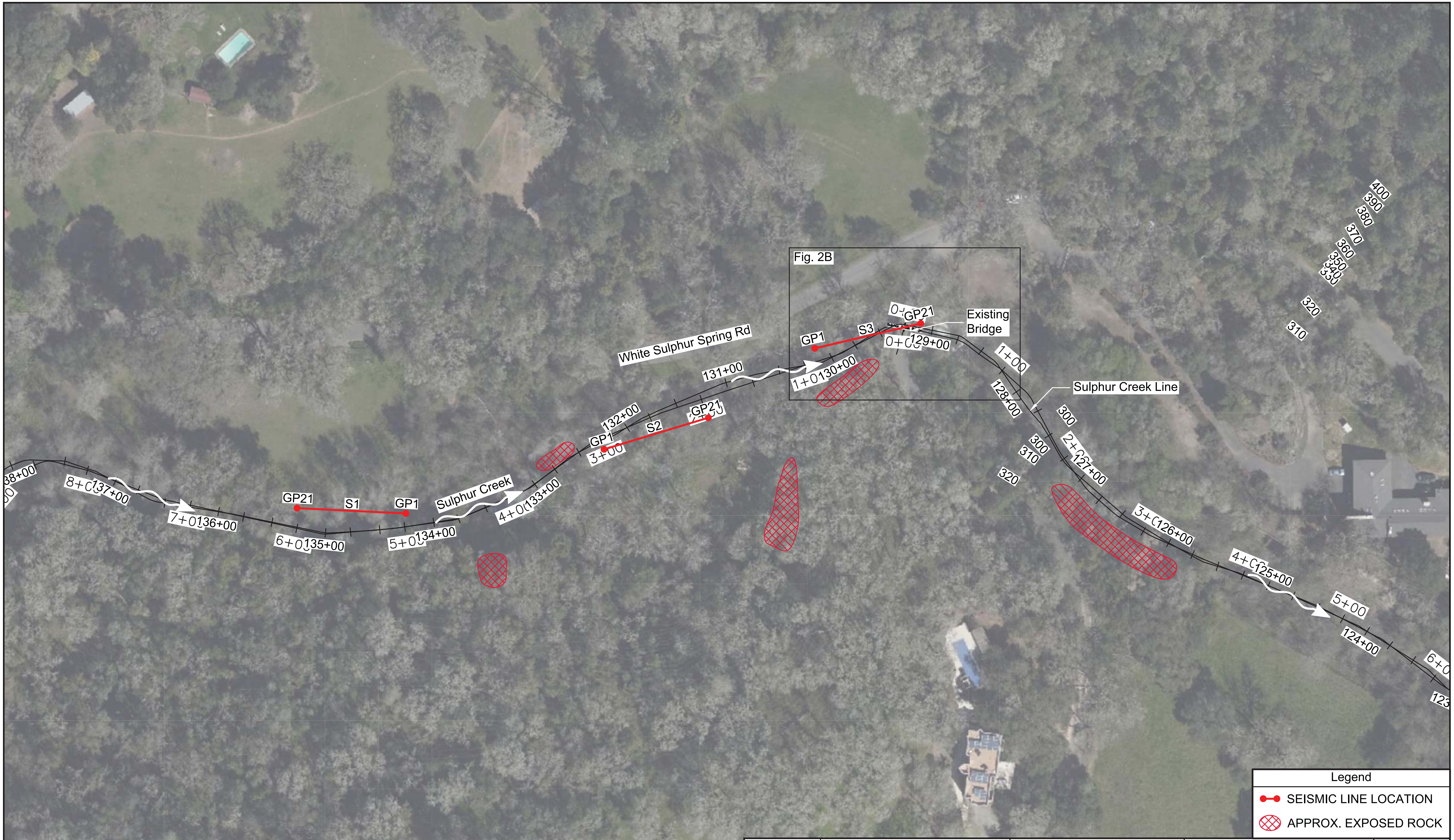
NCRCD-SULPHUR CREEK
FISH PASSAGE
(PROJECT #30144)

ST. HELENA, CALIFORNIA

Figure 1
Vicinity Map

Proj. No:	20-643.1
Scale:	1"=4,000'
Date:	2/4/21



Source:
Basemap: AutoCAD Civil3D Geolocation Tool, using Bing Maps

Topographic and plan files provided by WRA via electronic transfer on 08/12/2020 and 11/09/2020

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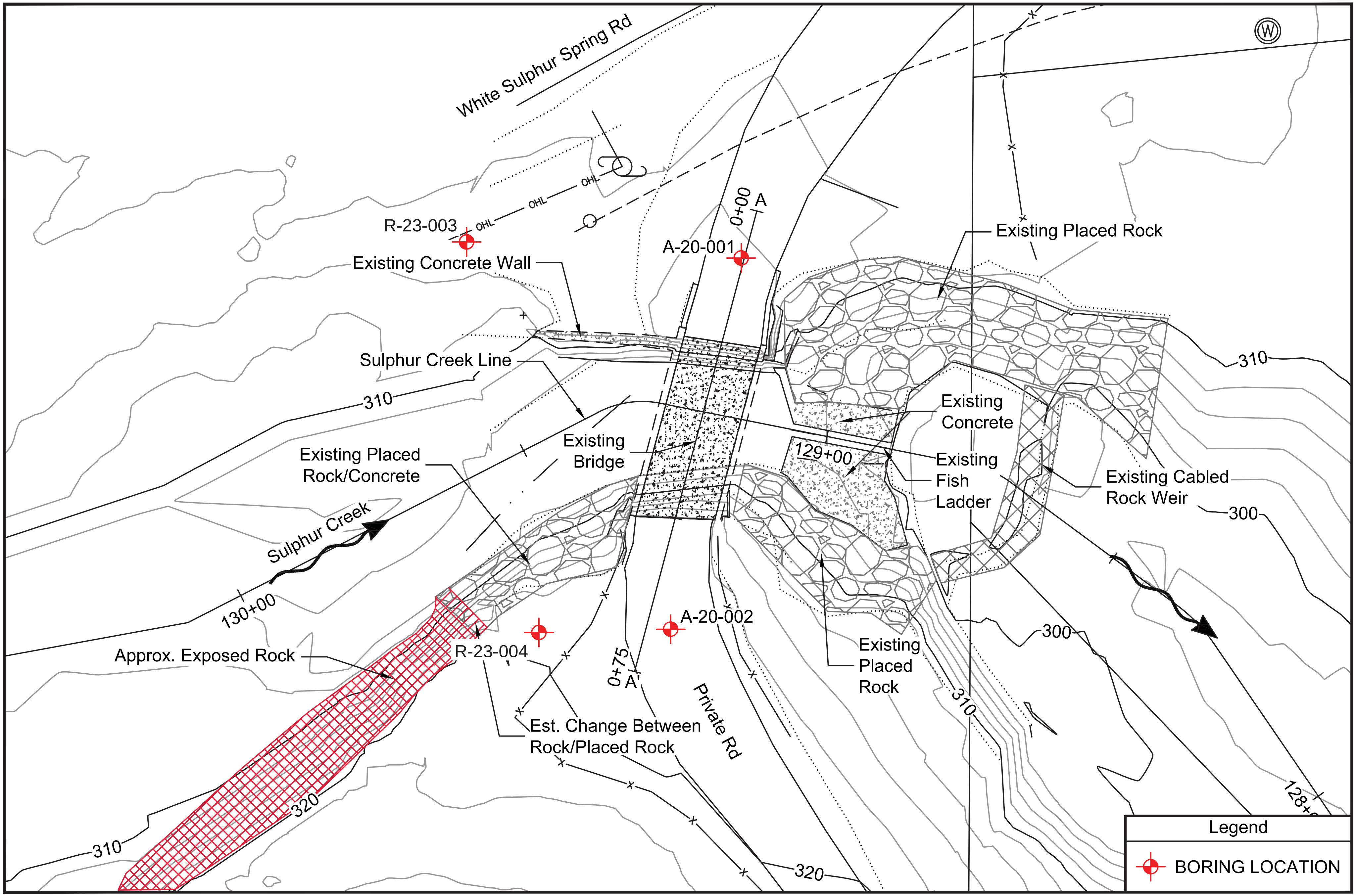
NCRCD-SULPHUR CREEK FISH PASSAGE (PROJECT #30144)

ST. HELENA, CALIFORNIA

Figure 2A
Exploration Map

Proj. No: 20-643.1
Scale: 1"=80'
Date: 2/3/23



North

Source:
Basemap: AutoCAD Civil3D Geolocation Tool, using Bing Maps

Topographic and plan files provided by WRA via electronic transfer on 08/12/2020 and 11/09/2020

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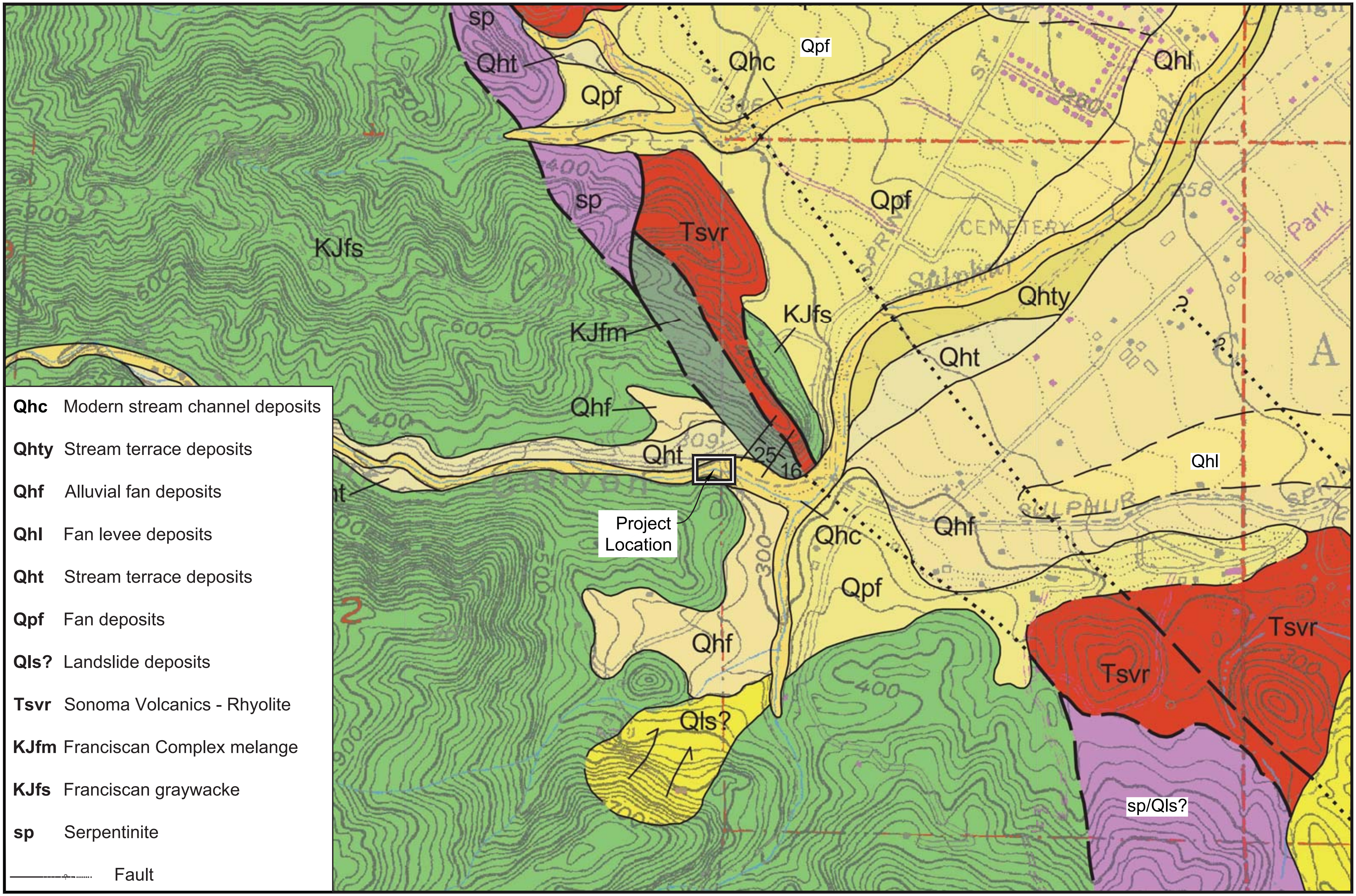
NCRCD-SULPHUR CREEK FISH PASSAGE (PROJECT #30144)

ST HELENA, CALIFORNIA

Figure 2B
Exploration Map

Proj. No:	20-643.1
Scale:	1"=20'
Date:	2/3/23



North

Source: Clahan, K.B., Wagner, D.L., Bezore, S.P., Sowers, J.M., and Witter, R.C.; Geologic map of the Rutherford 7.5-minute quadrangle, Sonoma and Napa counties, CA: A Digital database, v.1.0; Preliminary Geologic Maps; Scale: 1:24,000; California: California Geologic Survey, 2005.

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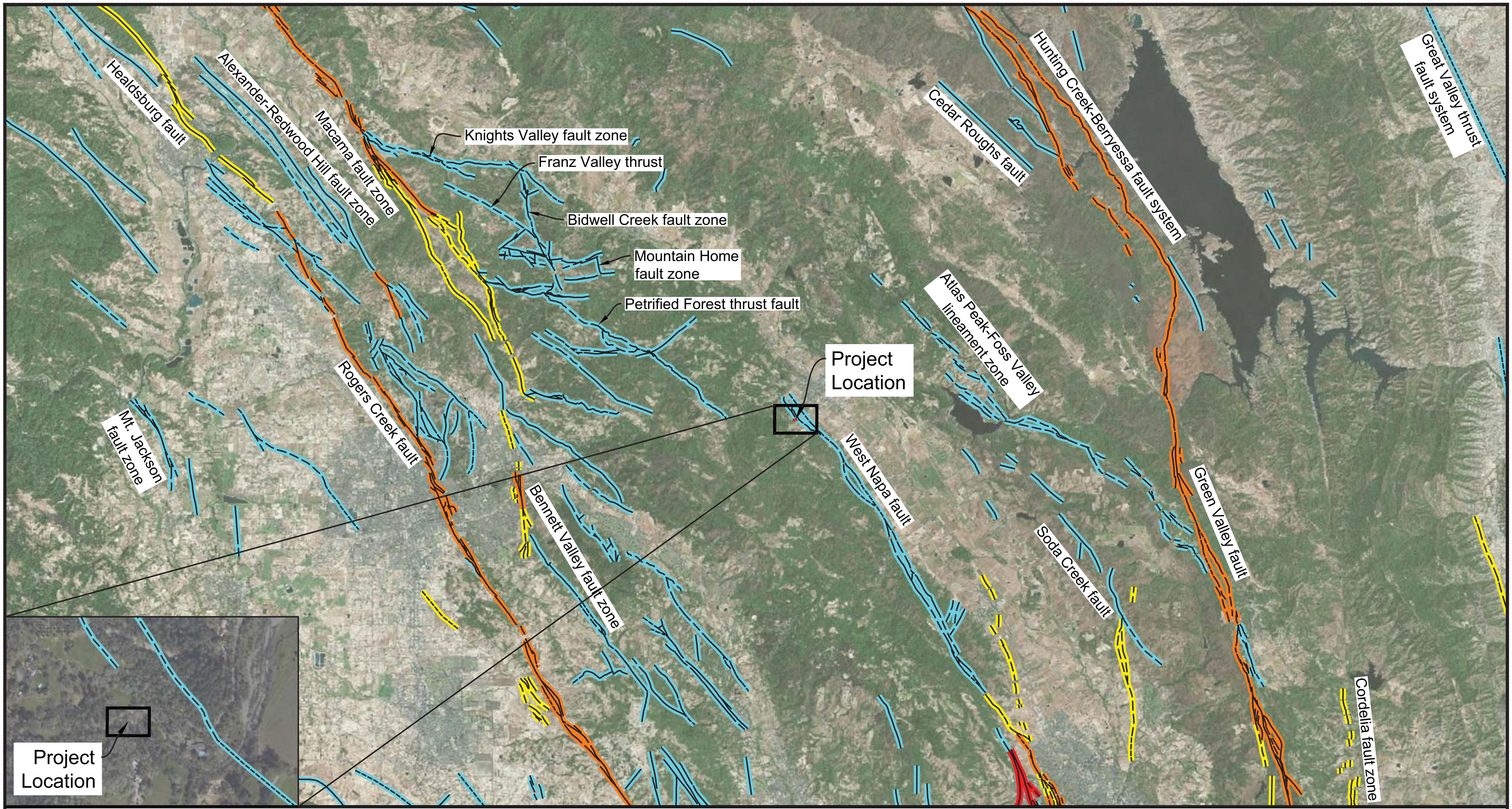
NCRCD-SULPHUR CREEK FISH PASSAGE (PROJECT #30144)

ST HELENA, CALIFORNIA

Figure 3
Geologic Map

Proj. No:	20-643.1
Scale:	1"=1,000'
Date:	2/4/21



LEGEND

Quaternary Fault (Age)

🟥 <150 years
🟧 <15,000 years
🟨 <130,000 years

Quaternary Fault (Age)

🟦 <750,000 years
🟦 <1.6 million years

Location

—— Well Constrained
- - - - Moderately Constrained
-------- Inferred

North

Source:
Basemap: AutoCAD Civil3D Geolocation Tool, using Bing Maps

Fault Data: USGS GIS Data

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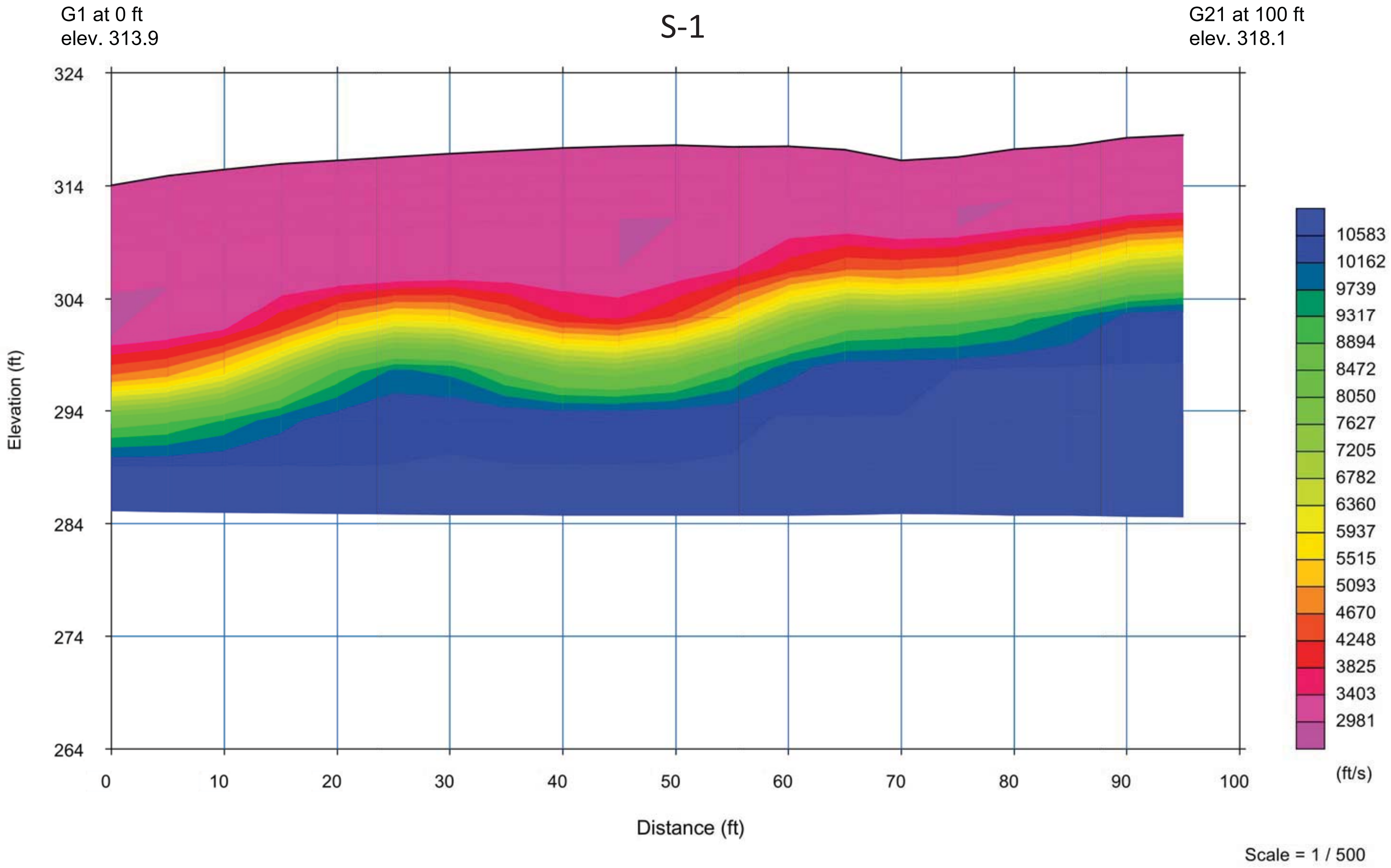
PASSAGE (PROJECT #30144)

ST HELENA, CALIFORNIA

Figure 4
Fault Map

Proj. No: 20-643.1
Scale: 1"=25,000'
Date: 2/4/21



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ST. HELENA, CALIFORNIA

Figure 5A
Seismic Refraction Profile 1 of 2

Proj. No:	20-643.1
Scale:	N/A
Date:	2/12/21

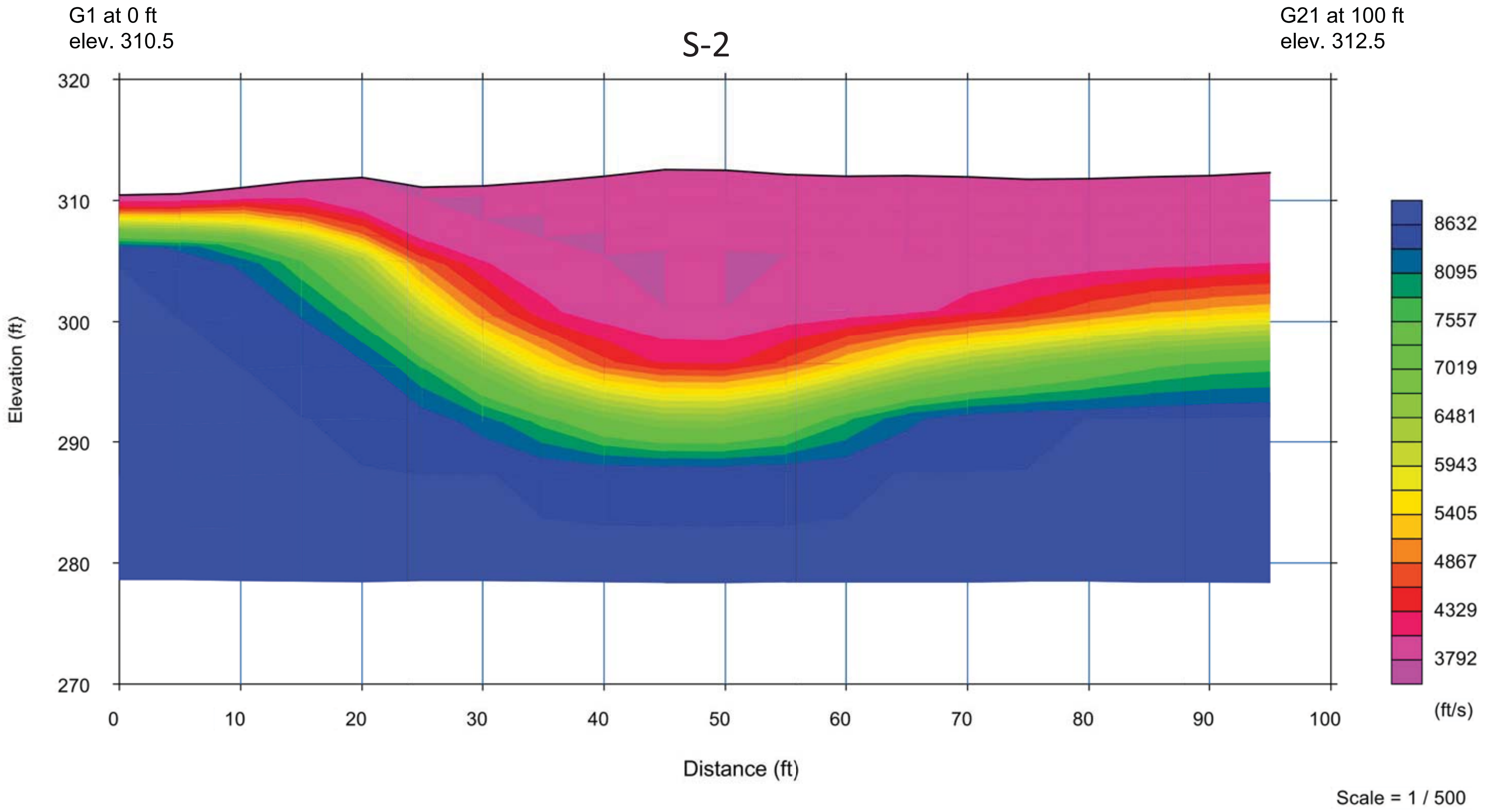

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Figure 5B
Seismic Refraction
Profile 2 of 2

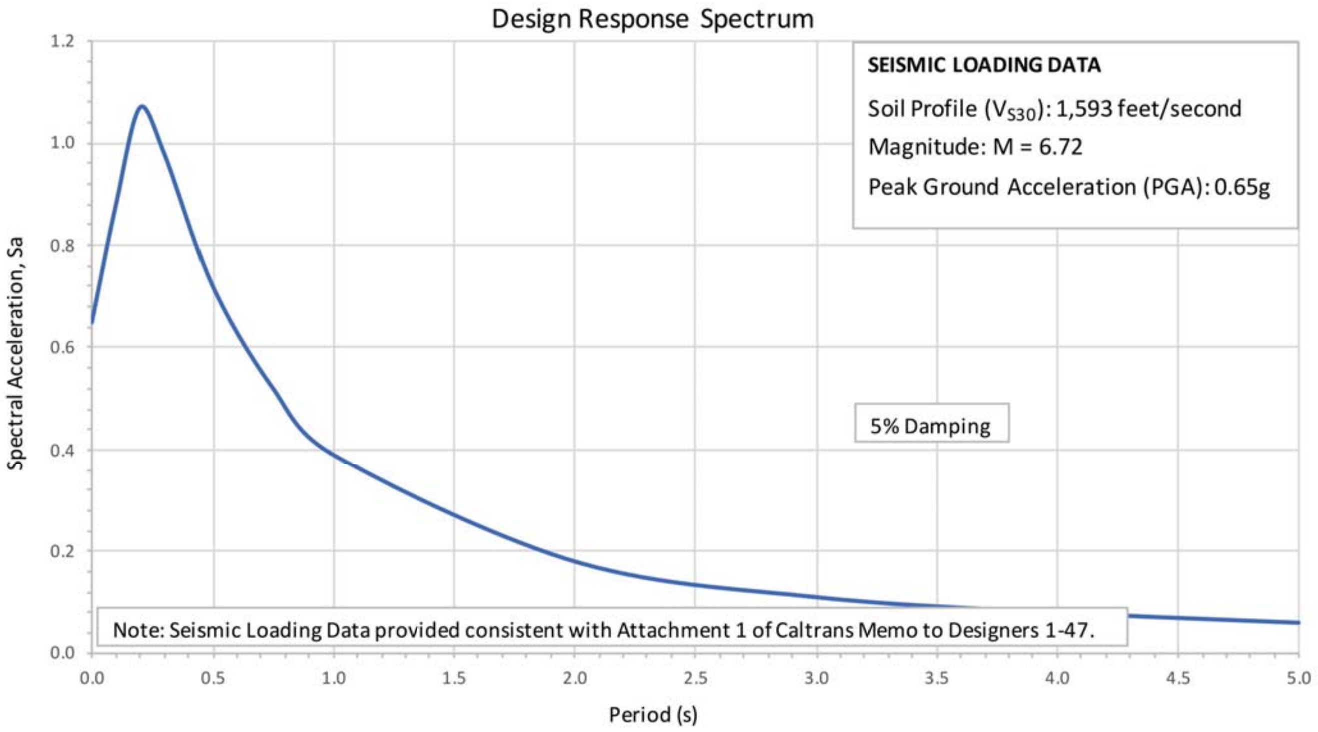
Proj. No: 20-643.1
Scale: N/A
Date: 2/12/21

APPENDIX II

Ground Motion Data Sheet

Caltrans ARS Online Version: V3.0.2

Period (s)	Spectral Acceleration, Sa (g)
0.000	0.65
0.100	0.88
0.200	1.07
0.300	0.98
0.500	0.72
0.750	0.52
1.000	0.39
2.000	0.18
3.000	0.11
4.000	0.08
5.000	0.06



SEISMIC LOADING DATA

Soil Profile (V_{S30}): 1,593 feet/second

Magnitude: M = 6.72

Peak Ground Acceleration (PGA): 0.65g

Note: Seismic Loading Data provided consistent with Attachment 1 of Caltrans Memo to Designers 1-47.

The Design Response Spectrum is developed using the 2014 probabilistic response spectrum obtained for the 5 percent probability of exceedance in 50 years (975-year return period) from the USGS Interactive Deaggregation web tool with

Site Latitude: 38.4879°
Site Longitude: -122.4816

Source:
https://arsonline.dot.ca.gov/, accessed 5/4/2023

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ST. HELENA, CALIFORNIA

Figure 6
Ground Motion Data Sheet

Proj. No: 20-643.1
Scale: N/A
Date: 5/5/23

APPENDIX III

Log of Test Borings

2021 Boring Logs

BENCH MARK

POINT	NORTHING	EASTING	ELEVATION	DESCRIPTION
53	1939977	6423854	312.45	CP 90D SHLDR/FNC
54	1939846	6423820	318.361	CP 90D EP/BRIDGE 54
55	1940014	6423796	311.988	SR SRHW TF 24IN UNDER 61NHW
1000	1939897	6423814	316.616	SVCP 60D
1001	1939904	6423859	313.642	SVCP 60D

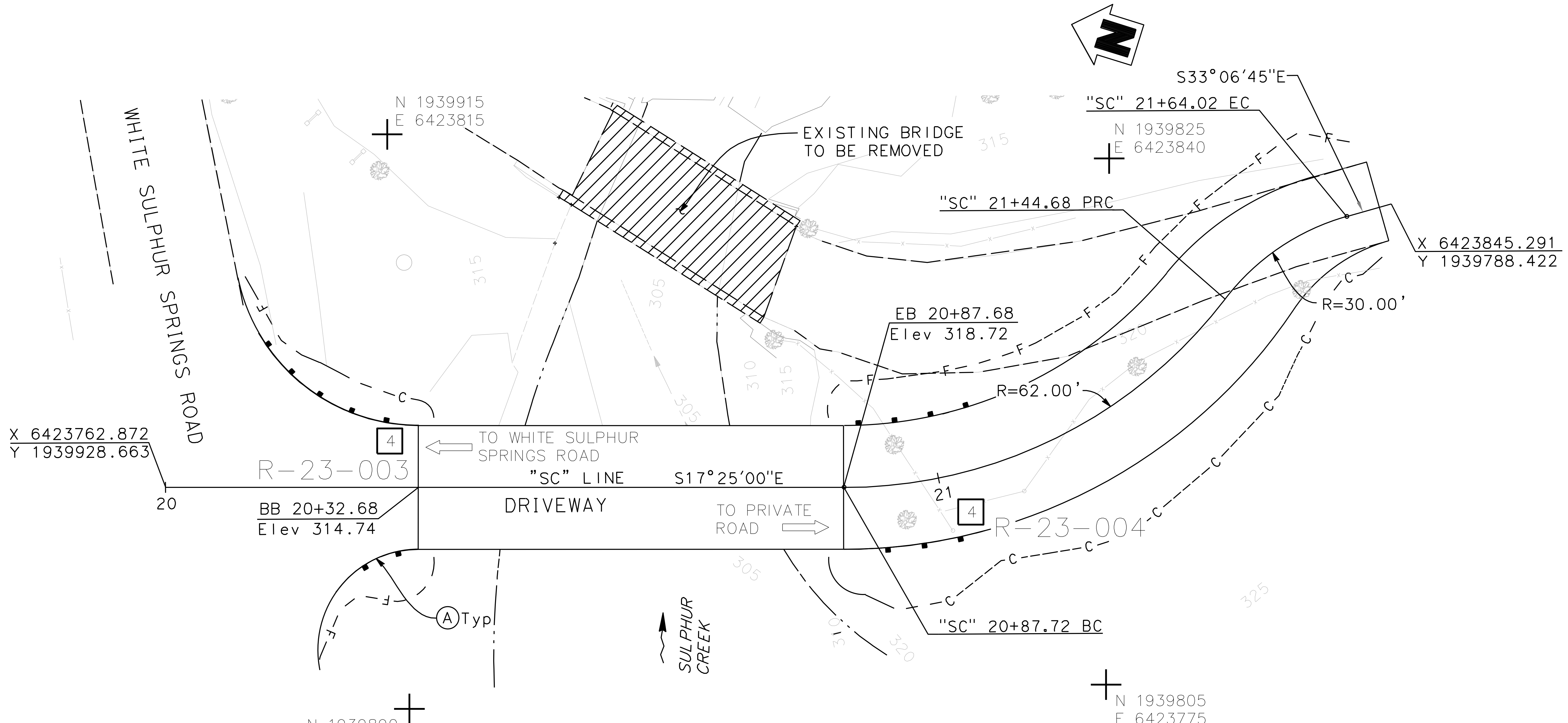
VERTICAL DATUM BASED ON NAVD '88

HORIZONTAL DATUM BASED ON NAD '83

PER TRIMBLE GPS OBSERVATIONS USING OPUS SOLUTION

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

See 2022 Standard Plans A10F and A10G for Soil Legend, and A10H for Rock Legend.



PLAN

1" = 10'

HOLE ID	NORTHING	EASTING	ALIGNMENT NAME	STATION AND OFFSET
R-23-003	1939904.3	6423772.1	"SC" LINE	20+29.00 6.0' Lt
R-23-004	1939834.3	6423785.4	"SC" LINE	21+03.00 5.2' Rt

wra Environmental Consultants
2169-G East Francisco Blvd.
San Rafael, CA 94901
(415) 454-8868 Phone
info@wra-ca.com

MARK THOMAS

SULPHUR CREEK FISH PASSAGE RESTORATION PROJECT

ST HELENA, CALIFORNIA
DESIGNED FOR:

NAPA RCD

65% DESIGN

NOT FOR CONSTRUCTION

ELLEN TAYLOR TIEDEMANN
No. 91681
CIVIL

DATE	ISSUES AND REVISIONS	NO.
02/10/2023	65% DESIGN PLANS	

PROJECT #30144
DRAWN BY: EK
CHECKED BY: JW
ORIGINAL DRAWING SIZE: 24 X 36

TEST BORING LAYOUT

SHEET

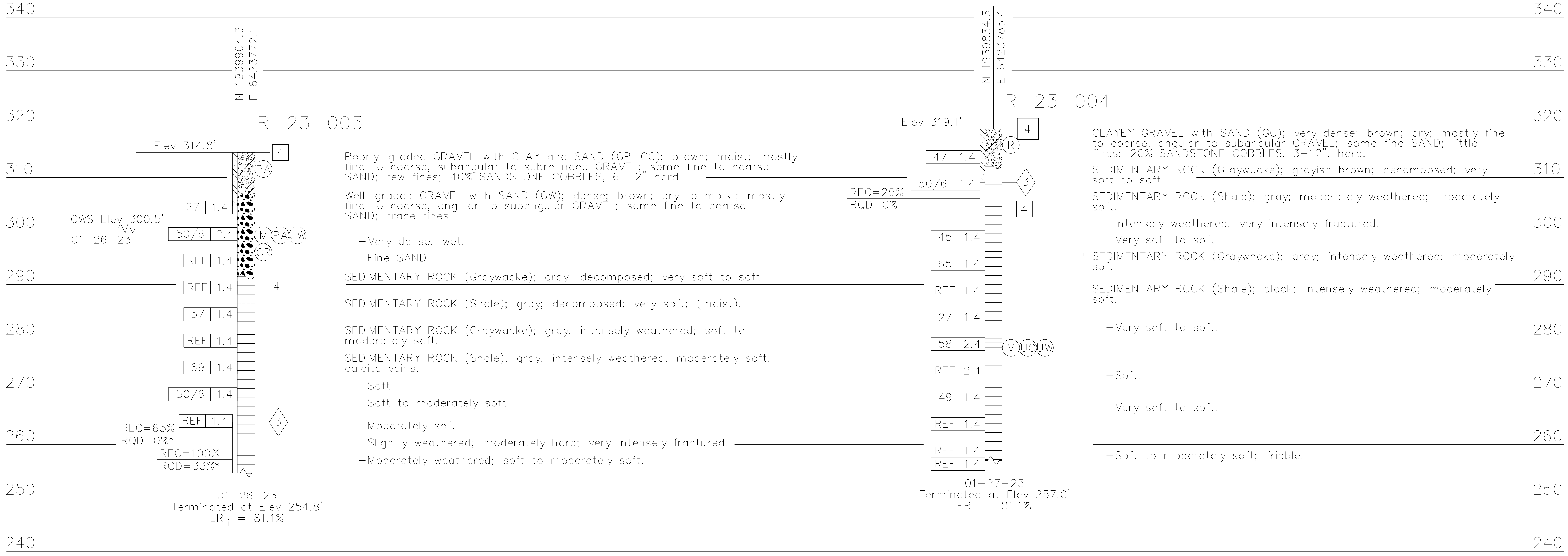
B1.7

FOR PLAN VIEW AND ADDITIONAL NOTES, SEE
"TEST BORING LAYOUT" SHEET

This LOTB sheet was prepared in accordance with the Caltrans Soil & Rock Logging, Classification, & Presentation Manual (2010 Edition).

See 2022 Standard Plans A10F and A10G for Soil Legend, and A10H for Rock Legend.

NOTES:
1. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with this Manual. 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.
2. Blow counts shown as "REF" where less than 0.5 feet of penetration were achieved in the first seating interval.
3. Density of soils shown in ( ) where estimated.
4. "2.4 inch sampler": ID = 2.4", OD = 3 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 are based solely on the visual practices described in the Caltrans Manual.
6. Groundwater elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
7. Groundwater elevation was not established in boring R-23-004 due to drilling method.
8. * Indicates that rock did not meet soundness requirement.



R-23-003

Poorly-graded GRAVEL with CLAY and SAND (GP-GC); brown; moist; mostly fine to coarse, subangular to subrounded GRAVEL; some fine to coarse SAND; few fines; 40% SANDSTONE COBBLES, 6-12" hard.

Well-graded GRAVEL with SAND (GW); dense; brown; dry to moist; mostly fine to coarse, angular to subangular GRAVEL; some fine to coarse SAND; trace fines.

-Very dense; wet.

-Fine SAND.

SEDIMENTARY ROCK (Graywacke); gray; decomposed; very soft to soft.

SEDIMENTARY ROCK (Shale); gray; decomposed; very soft; (moist).

SEDIMENTARY ROCK (Graywacke); gray; intensely weathered; soft to moderately soft.

SEDIMENTARY ROCK (Shale); gray; intensely weathered; moderately soft; calcite veins.

-Soft.

-Soft to moderately soft.

-Moderately soft

-Slightly weathered; moderately hard; very intensely fractured.

-Moderately weathered; soft to moderately soft.

R-23-004

CLAYEY GRAVEL with SAND (GC); very dense; brown; dry; mostly fine to coarse, angular to subangular GRAVEL; some fine SAND; little fines; 20% SANDSTONE COBBLES, 3-12", hard.

SEDIMENTARY ROCK (Graywacke); grayish brown; decomposed; very soft to soft.

SEDIMENTARY ROCK (Shale); gray; moderately weathered; moderately soft.

-Intensely weathered; very intensely fractured.

-Very soft to soft.

SEDIMENTARY ROCK (Graywacke); gray; intensely weathered; moderately soft.

SEDIMENTARY ROCK (Shale); black; intensely weathered; moderately soft.

-Very soft to soft.

-Soft.

-Very soft to soft.

-Soft to moderately soft; friable.

PROFILE
Vert: 1"=10'

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(415) 454-8868 Phone
info@wra-ca.com

MARK THOMAS

SULPHUR CREEK FISH PASSAGE RESTORATION PROJECT

ST HELENA, CALIFORNIA
DESIGNED FOR:

NAPA RCD

65% DESIGN

NOT FOR CONSTRUCTION

REGISTERED PROFESSIONAL ENGINEER
ELLEN TAYLOR TIEDEMANN
No. 91681
CIVIL
STATE OF CALIFORNIA

DATE	ISSUES AND REVISIONS	NO.
02/10/2023	65% DESIGN PLANS	

PROJECT #30144
DRAWN BY: EK
CHECKED BY: JW
ORIGINAL DRAWING SIZE: 24 X 36

LOG OF TEST BORINGS

SHEET
B1.8

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (CTM 216)
CR	Corrosion, Sulfates, Chlorides (CTM 643, CTM 417, CTM 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DR	Drained Residual Shear Strength (ASTM D 6467)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeability (CTM 220)
PA	Particle Size Analysis (ASTM D 422)
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89, AASHTO T 90)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SW	Swell Potential (ASTM D 4546)
UC	Unconfined Compression - Soil (ASTM D 2166) Unconfined Compression - Rock (ASTM D 7012-C)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 7263)

SAMPLER GRAPHIC SYMBOLS

- ☒ Standard Penetration Test (SPT)
- ⧗ Standard California Sampler (ID 2.0 in.)
- ⧗ Modified California Sampler (ID 2.5 in.)
- ▯ Shelby Tube
- ▯ Piston Sampler
- ▯ NX Rock Core
- ▯ HQ Rock Core
- ▨ Bulk Sample
- ◪ Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- ▽ First Water Level Reading (during drilling)
- ▼ Static Water Level Reading (short-term)
- ▼ Static Water Level Reading (long-term)

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010) with Errata Sheet (2015).

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Boring Record Legend

Soil Legend

Sheet 1 of 2

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} (blows / 12 inches)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

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Boring Record Legend

Soil Legend

Sheet 2 of 2

ROCK GRAPHIC SYMBOLS

☒	IGNEOUS ROCK
▤	SEDIMENTARY ROCK
▧	METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 ft - 10 ft
Thickly bedded	1 ft - 3 ft
Moderately bedded	4 in - 1 ft
Thinly bedded	1 in - 4 in
Very thinly bedded	1/4 in - 1 in
Laminated	< 1/4 in

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

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

PERCENT CORE RECOVERY (REC)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

ROCK QUALITY DESIGNATION (RQD)

$$\frac{\Sigma \text{ Length of intact core pieces > 4 in.}}{\text{Total length of core run (in.)}} \times 100$$

Note: RQD* indicates soundness criteria not met

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/16 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light manual pressure.

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Core lengths greater than 3 ft.
Slightly Fractured	Core lengths mostly from 1 ft. to 3 ft.
Moderately Fractured	Core lengths mostly from 4 in. to 1 ft.
Intensely Fractured	Core lengths mostly from 1 in. to 4 in.
Very Intensely Fractured	Mostly chips and fragments.

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).

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Boring Record Legend

Rock Legend

Sheet 1 of 1

LOG OF BORING A-21-001

PROJECT NO: 20-643.1
PROJECT: NCRCD-Sulphur Creek Fish Passage (Project #30144)
LOCATION: St. Helena
COUNTY: NAP
CLIENT: Mark Thomas
LOGGED BY: KBH
DEPTH OF BORING: 28.40 (ft)

BEGIN DATE: 01/05/2021
COMPLETION DATE: 01/05/2021
SURFACE ELEVATION: 316.30 (ft)
SURFACE CONDITION: Asphalt
WATER DEPTH: 17.5 ft
READING TAKEN: 01/05/21
HAMMER EFFICIENCY: 89.3 (%)

DRILLING CONTRACTOR: GeoEx Subsurface Exploration
DRILLING METHOD: SS Augers 4.0"
DRILL RIG: CME 55 (Truck Mounted)
HAMMER TYPE: Automatic; 140 lbs; 30 in. drop
SAMPLER TYPE & SIZE: MCAL (2.4" ID), SPT (1.4" ID)
BOREHOLE DIAMETER: 4.0 in.
BACKFILL METHOD: Neat Cement Grout

FIELD											LABORATORY							
ELEVATION (ft)	DEPTH (ft)	SAMPLE	SAMPLE NO	BLOWS PER 6 IN.	BLOWS PER FOOT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	RECOVERY(%)	RQD (%)	PLASTIC LIMIT	LIQUID LIMIT	MOISTURE (%)	D. DENSITY (PCF)	% PASSING 200 SIEVE	DRILL METHOD	CASING DEPTH	REMARKS
316								ASPHALT .										AC=1"
315	1							AGGREGATE BASE . CLAYEY GRAVEL with SAND (GC); very dense; gray; dry; mostly coarse to fine GRAVEL; little coarse to fine SAND; little fines; [FILL].										AB=3" Chatter from gravels observed 0-5'
314	2																	
313	3																	
312	4																	
311	5	⧓	1	50/5	REF			CLAYEY SAND (SC); very dense; gray; dry; mostly medium to fine SAND; some fines; moderate cementation.	0									
310	6																	
309	7																	
308	8																	
307	9																	
306	10	⧓	2	21 50	50/6			coarse to fine SAND; trace fine, subrounded GRAVEL; moderate cementation	100									Sampler rebounding
305	11																	Grinding observed at 11'
304	12																	
303	13	⧓	3	25 44 44	88			dry to moist; few coarse to fine, subrounded GRAVEL	67									
302	14																	
301	15	⧓	4	22 50	50/6				25									Sampler reboundin
300	16							Poorly-graded GRAVEL with CLAY (GP-GC); very dense; gray; dry; mostly coarse, subangular GRAVEL; few medium to fine SAND; few fines.										Driller notes harder drilling 16-17'.
299	17 ▽																	
298	18 ▼	☒	5	9 28 30	58			Sedimentary (Shale); gray with reddish oxidation; very intensely weathered; soft to moderately soft; very intensely to intensely fractured; (wet).	61									
297	19												11.9	127.7				
296	20	☒	6	2 12 17	29			decomposed; soft	17									
295	21																	
294	22																	Grinding observed 23-25'.
293	23																	Hole caved to 20' using SSA; switch to mud rotary at 25'.
292	24																	
291	25	☒	7	21 40 50	90			intensely weathered	56									Soil pH: 6.60 Min. Resistivity: 3,220 ohm-c
290	26												9.3	132.6				Chloride: 2.9 ppm
289	27							Sedimentary (Graywacke); gray; intensely weathered; soft; (moist).										Sulfate: 11.7 ppm Slow drilling and rig shaking observed at 27'; auger refusal at 28'.
288	28	☒	8	50/5	REF				80				9.2	136.1				

Bottom of borehole at 28.4 ft bgs

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

Crawford & Associates, Inc.
1100 Corporate Way, Suite 230
Sacramento, CA 95831
(916) 455-4225

PROJECT NO: 20-643.1
PROJECT: NCRCD-Sulphur Creek Fish Passage (Project #30144)
BORING: A-21-001
ENTRY BY: KBH
CHECKED BY: ETT

LOG OF BORING A-21-002

PROJECT NO: 20-643.1
PROJECT: NCRCD-Sulphur Creek Fish Passage (Project #30144)
LOCATION: St. Helena
COUNTY: NAP
CLIENT: Mark Thomas
LOGGED BY: KBH
DEPTH OF BORING: 20.25 (ft)

BEGIN DATE: 01/05/2021
COMPLETION DATE: 01/05/2021
SURFACE ELEVATION: 318.90 (ft)
SURFACE CONDITION: Asphalt
WATER DEPTH: Not Encountered
READING TAKEN: N/A
HAMMER EFFICIENCY: 89.3 (%)

DRILLING CONTRACTOR: GeoEx Subsurface Exploration
DRILLING METHOD: SS Augers 4.0", Mud Rotary 4.0"
DRILL RIG: CME 55 (Truck Mounted)
HAMMER TYPE: Automatic; 140 lbs; 30 in. drop
SAMPLER TYPE & SIZE: MCAL (2.4" ID), SPT (1.4" ID)
BOREHOLE DIAMETER: 4.0 in.
BACKFILL METHOD: Neat Cement Grout

FIELD											LABORATORY							
ELEVATION (ft)	DEPTH (ft)	SAMPLE	SAMPLE NO	BLOWS PER 6 IN.	BLOWS PER FOOT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	RECOVERY(%)	RQD (%)	PLASTIC LIMIT	LIQUID LIMIT	MOISTURE (%)	D. DENSITY (PCF)	% PASSING 200 SIEVE	DRILL METHOD	CASING DEPTH	REMARKS
								ASPHALT . AGGREGATE BASE . CLAYEY SAND (SC); dense; light brown; dry; mostly medium to fine SAND; trace fine, subround GRAVEL; little medium plasticity fines.										AC=3" AB=3" Driller notes gravelly drilling 0-5'
318	1																	
317	2																	
316	3																	
315	4																	
314	5		1	7 17 22	39				67									Driller notes harder drilling 5-10', grinding observed
313	6																	
312	7																	
311	8							Sedimentary (Graywacke); gray; intensely to moderately weathered; moderately soft.										
310	9																	
309	10		2	50	REF				0									Sampler rebounding
308	11																	
307	12																	
306	13																	
305	14																	
304	15		3	50/4	REF				100				5.5					Driller notes harder drilling 10-15'
303	16																	
302	17																	
301	18		4	38 50/4	50/4			soft	10									
300	19																	
299	20		5	50/3	REF			moderately weathered Bottom of borehole at 20.2 ft bgs	33									Auger Refusal
298	21																	
297	22																	
296	23																	
295	24																	
294	25																	
293	26																	
292	27																	
291	28																	

APPENDIX IV

Laboratory Test Results

Project Name: NCRCD Sulphur Creek Fish Passage
CAInc File No: 20-643.1
Date: 3/6/23
Technician: 2/6/2023-2/28/23

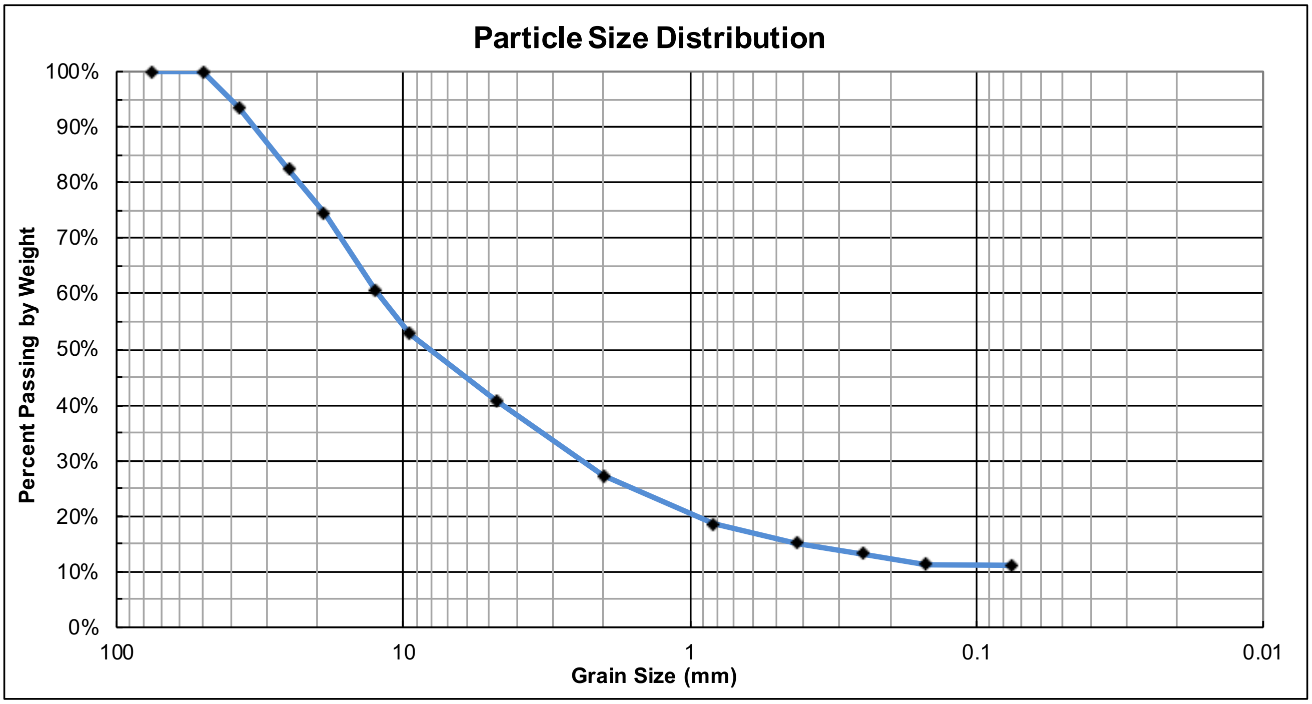
MOISTURE-DENSITY TESTS - D2216/D7263

	1	2	3	4	5
Sample No.	R-23-003-2A	R-22-004-7A			
USCS Symbol	GW	Shale			
Depth (ft.)	15.5	41			
Sample Length (in.)	5.410	5.879			
Diameter (in.)	2.380	2.402			
Sample Volume (ft^3)	0.01393	0.01541			
Total Mass Soil+Tube (g)	1208.8	1013.2			
Mass of Tube (g)	285.5	0.0			
Tare No.	X11	H6			
Tare (g)	115.2	13.3			
Wet Soil + Tare (g)	636.9	101.7			
Dry Soil + Tare (g)	589.9	94.8			
Dry Soil (g)	474.7	81.5			
Water (g)	47.0	6.9			
Moisture (%)	**9.9**	**8.5**			
Dry Density (pcf)	**133.0**	**133.5**			

Notes:

Project Name: NCRCD Sulphur Creek Fish Passage
CAInc File No: 20-643.1
Date: 2/15/23
Technician: ZZZ
Sample ID: R-23-003-Bulk
Depth (ft): 0
USCS Classification: Poorly-graded Gravel with Clay and Sand (GP-GC)

ASTM 6913 - Method A



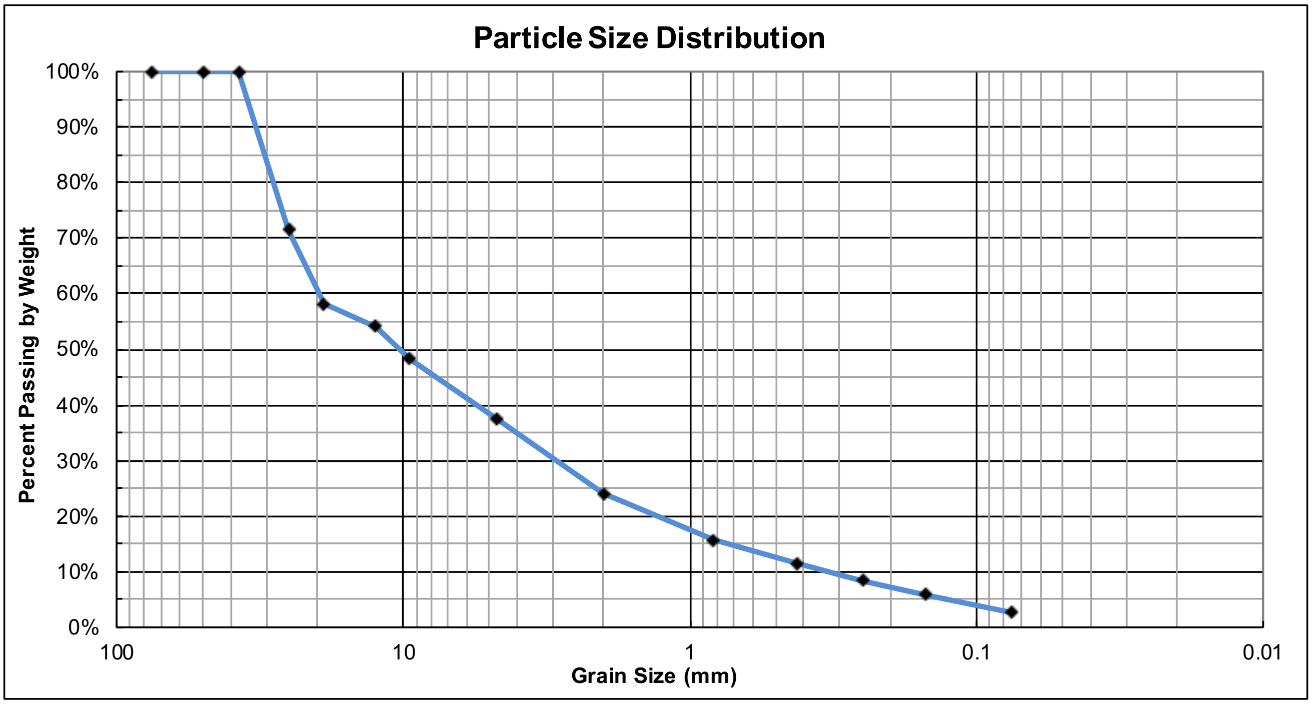
% Cobble	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
	25	34	14	12	4	
0	**59**		**30**			**11**

		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	57.4	94%
		1"	25.0	157.8	82%
		3/4"	19.0	228.0	75%
	Fine	**1/2"**	12.5	353.0	61%
		3/8"	9.50	421.2	53%
		#4	4.75	529.6	41%
Sand	Coarse	**#10**	2.00	651.0	27%
	Medium	**#20**	0.825	728.4	19%
		#40	0.425	758.3	15%
	Fine	**#60**	0.250	775.9	13%
		#100	0.150	793.1	11%
Silt/Clay		**#200**	0.075	794.5	11%

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

Project Name: NCRCD Sulphur Creek Fish Passage
CAInc File No: 20-643.1
Date: 2/15/23
Technician: ZZZ
Sample ID: R-23-003-2A
Depth (ft): 15.5
USCS Classification: Well-graded Gravel with Sand (GW)

ASTM 6913 - Method A



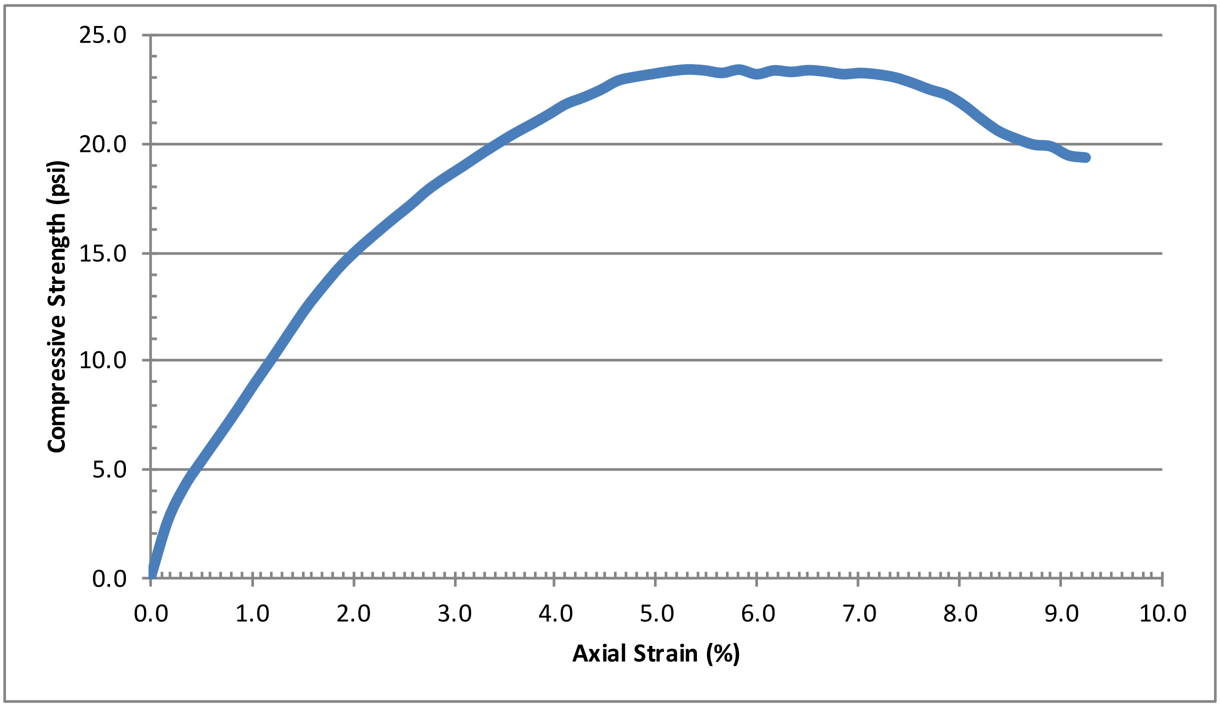
% Cobble	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
	42	21	13	12	9	
0	**63**		**34**			**3**

		Sieve #	Opening mm	Cummulative Mass Retained (g)	% Passing %
Cobbles		**3"**	75	0.0	100%
Gravel	Coarse	**2"**	50	0.0	100%
		1-1/2"	37.5	0.0	100%
		1"	25.0	124.5	72%
		3/4"	19.0	182.9	58%
	Fine	**1/2"**	12.5	200.7	54%
		3/8"	9.50	226.0	48%
		#4	4.75	273.6	37%
Sand	Coarse	**#10**	2.00	332.3	24%
	Medium	**#20**	0.825	369.1	16%
		#40	0.425	387.2	12%
	Fine	**#60**	0.250	400.9	8%
		#100	0.150	411.9	6%
Silt/Clay		**#200**	0.075	426.1	3%

Coefficient of Uniformity	Coefficient of Curvature
Cu = 58.2	Cc = 1.5

Project Name: NCRCD Sulphur Creek Fish Passage
CAInc File No: 20-643.2
Date: 2/28/23
Technician: CAP
Sample ID: R-23-004-7A Depth (ft): 41.0
USCS Classification: Shale

UNCONFINED COMPRESSION TEST - D2166



Dry Density (pcf)	**133.5**
Water Content (%)	**8.5**
Unconfined Compressive Strength (psi)	**23.4**
Unconfined Compressive Strength (psf)	**3370**
Average Height (in)	5.879
Average Diameter (in)	2.402
Rate of strain (%)	1.0
Strain at Failure (%)	5.3

Notes:

RESISTANCE VALUE

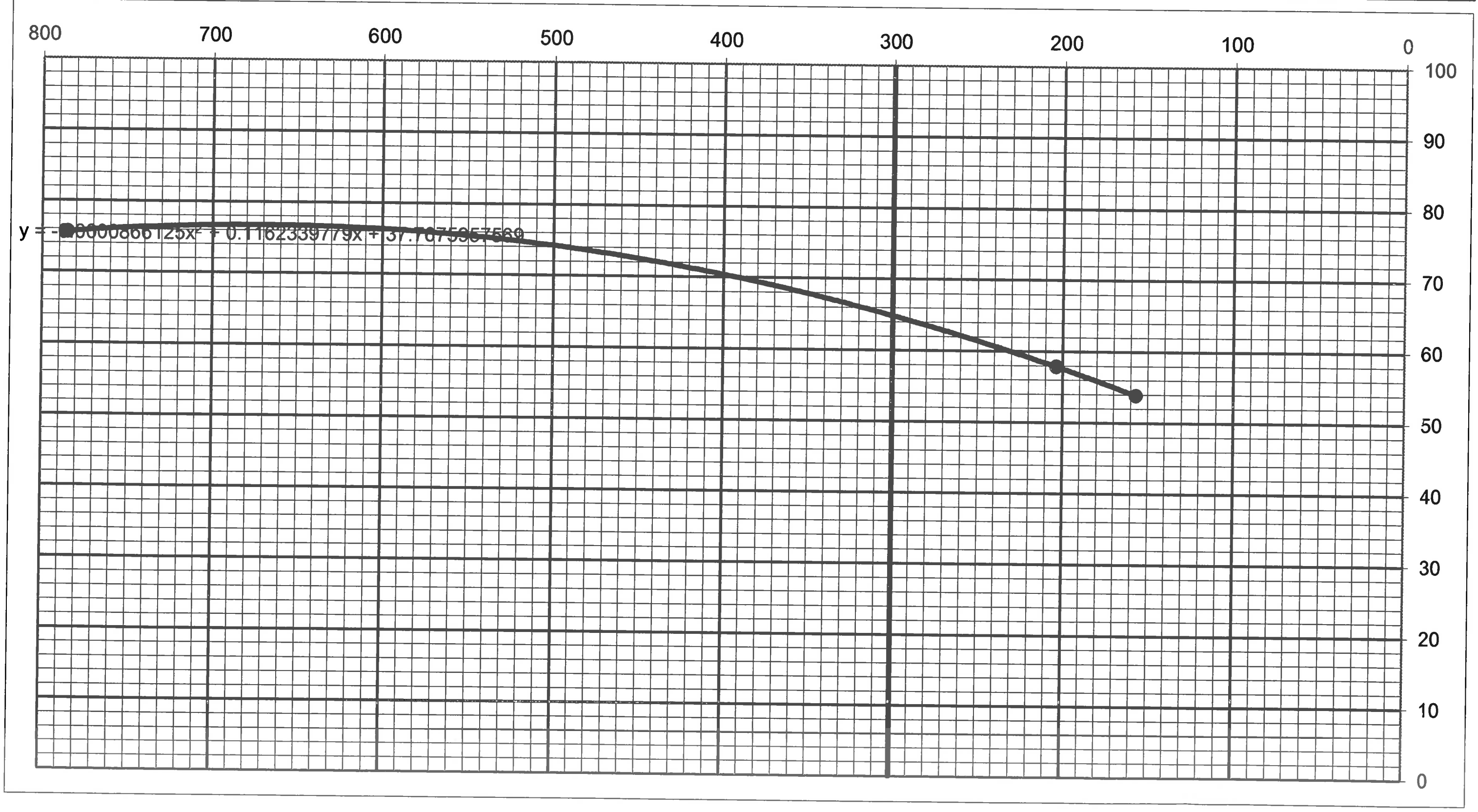
California Test Method No. 301

Job Number: 4151-001.00 **Date Tested:** 2/9/2023 **R-value:** 65

Project: Crawford (20-463.1 Sulphur Creek Fish Passage Project) **Sample :** R-23-004

Classification of Material: SC, Clayey Sand with Gravel, Dark Brown **Technician:** DS

Initial Sample Weight	1100	1060	1080		
Mold Number	E	F	D		
Air Pressure-PSI	350	350	350		
Initial Moisture %	15.7	15.7	15.7		
Water Added ml/g	-20	-10			
Water Added %	-2.1	-1.1			
Final Moist %	13.6	14.6	15.7		
Soil + Mold Weight-Grams	3107.2	3123.8	3150.2		
Mold Weight-Grams	2029	2067	2073		
Soil Weight-Grams	1078.2	1056.8	1077.2		
Height of Sample-Inches	2.48	2.43	2.47		
Density-PCF	116.0	115.0	114.2		
Dial Reading (x.0001 inches)	108	33	14		
Expansion Pressure (psf)	468	143	61		
Stabilometer at 1000 lbs.	13	22	24		
2000 lbs.	26	44	49		
Displacement	4.15	4.49	4.85		
Exudation Pressure-Lbs	9860	2560	1970		
Exudation-PSI	785	204	157		
R-Value Calculated	76	59	54		
Corrected R-Value	76	58	54		



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

Date Reported 02/10/2023
Date Submitted 02/06/2023

To: Kennedy Hauder
Crawford & Associates, Inc.
4701 Freeport Blvd
Sacramento, CA 95822

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

The reported analysis was requested for the following location:
Location : 20-643.1 Site ID : R-23-003-2A&3A(16-20.5').
Thank you for your business.

* For future reference to this analysis please use SUN # 88989-184868.

EVALUATION FOR SOIL CORROSION

Soil pH	7.54	
Minimum Resistivity	2.60 ohm-cm (x1000)	
Chloride	1.8 ppm	00.00018 %
Sulfate	7.6 ppm	00.00076 %

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

Project Name: NCRCD-Sulphur Creek Fish Passage (Project #30144)
CAInc File No: 20-643.1
Date: 1/26/20
Technician: OMR

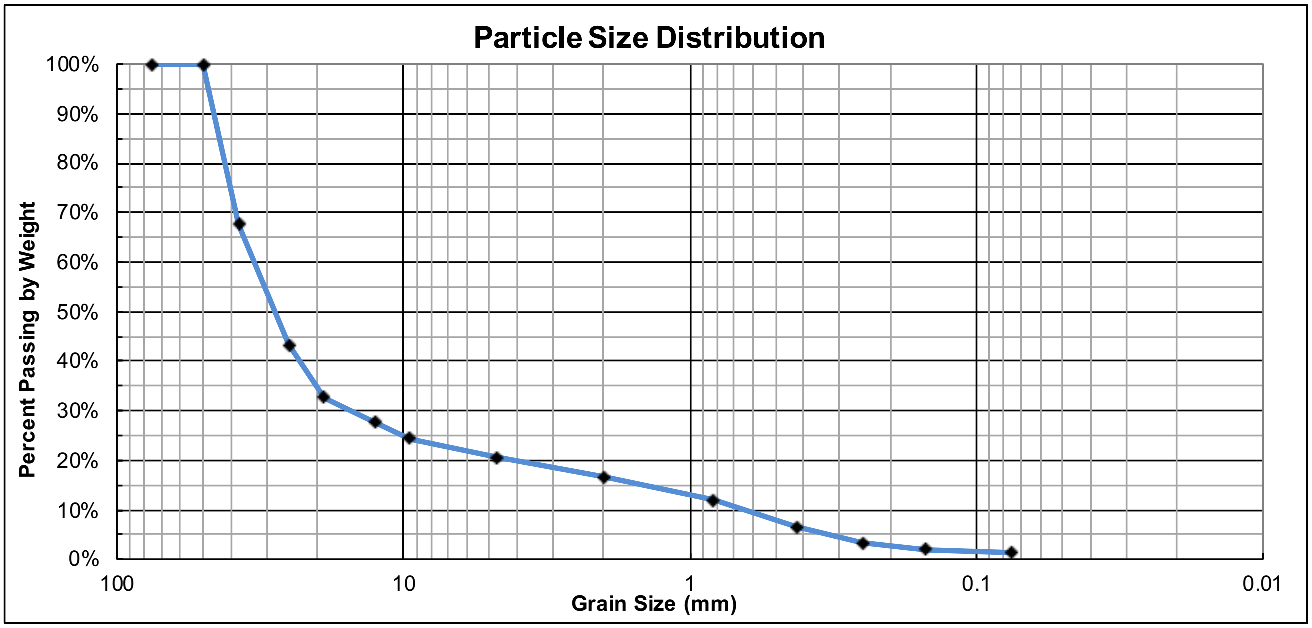
MOISTURE-DENSITY TESTS - D2216/D7263

	1	2	3	4	5
Sample No.	A-20-001-5A	A-20-001-7A	A-20-001-8A	A-21-002-3A	
USCS Symbol	SC	SC	GP-GC	SC	
Depth (ft.)	18.5	26	28	15	
Sample Length (in.)	2.952	4.948	3.005	4.918	
Diameter (in.)	1.385	1.428	1.402	1.408	
Sample Volume (ft^3)	0.00257	0.00458	0.00268	0.00443	
Total Mass Soil+Tube (g)	166.9	423.5	312.3	372.2	
Mass of Tube (g)	0.0	122.2	131.4	130.4	
Tare No.	D6	D15	155	G24	
Tare (g)	13.7	13.9	14.1	13.7	
Wet Soil + Tare (g)	73.9	67.4	71.0	76.8	
Dry Soil + Tare (g)	67.5	62.8	66.3	73.5	
Dry Soil (g)	53.8	48.9	52.2	59.9	
Water (g)	6.4	4.6	4.8	3.3	
Moisture (%)	**11.9**	**9.3**	**9.2**	**5.5**	
Dry Density (pcf)	**127.7**	**132.6**	**136.1**	**114.0**	

Notes:

Project Name: NCRC Sulphur Creek Fish Passage (Project #30144)
CAInc File No: 20-643.1
Date: 1/28/21
Technician: O.R.
Sample ID: Channel Bulk
Depth (ft): Channel
USCS Classification: Poorly Graded Gravel with Sand (GP)

ASTM 6913 - Method A



% Cobble	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
	67	12	4	11	5	
0	**79**		**20**			**1**

		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	379.3	68%
		1"	25.0	666.5	43%
		3/4"	19.0	790.5	33%
	Fine	**1/2"**	12.5	850.6	28%
		3/8"	9.50	888.6	24%
		#4	4.75	931.6	21%
Sand	Coarse	**#10**	2.00	980.1	17%
	Medium	**#20**	0.825	1033.2	12%
		#40	0.425	1098.4	6%
	Fine	**#60**	0.250	1136.1	3%
		#100	0.150	1150.6	2%
		#200	0.075	1158.3	1%

Coefficient of Uniformity	Coefficient of Curvature	50% of Cumulative Mass
Cu = 49.3	Cc = 10.7	D_{50} = 28.46

APPENDIX V

Foundation Design Calculations

INTRODUCTION

This appendix presents our foundation design calculations that include geotechnical design parameters, assumptions, methodology, and summaries the results of our pile foundation analysis. The results of our pile foundation analysis consist of compression resistance and lateral resistance. Our pile 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**
- **Shear Wave Velocity**
- **Deep Foundations (Bridge)**
  - *Compression Resistance*
  - *Lateral Resistance*

GEOTECHNICAL DESIGN PARAMETERS

The idealized geotechnical engineering properties and strength 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 project 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 305 feet; 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 and Table V-2.

Table V-1: Idealized Geotechnical Parameters – Abutment 1

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					
315 to 305	Poorly-graded Gravel with Clay and Sand (GP-GC) and Well-graded Gravel with Sand (GW)	36 to 100	Gravel	Sand (Reese)	145	37	--	--	225
305 to 292					83				125
292 to 265	Sedimentary Rock (Graywacke and Shale) Decomposed to Mod. Weathered, Mod. Soft to V. Soft	57 to 100	Clay	Stiff Clay (Without Water)	83	--	2,300	0.005	--
265 to 255					83		4,000	0.004	

Notes: Elevations are based on project datum provided by Mark Thomas.
In soil layer, the buoyant unit weight should be used below design groundwater (elev. 305 feet)
For design scour consideration, no soil/rock support is available above the scour elevation.

Table V-2: Idealized Geotechnical Parameters – Abutment 2

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					
319 to 313	Clayey Gravel with Sand (GC)	64	Gravel	Sand (Reese)	145	37	--	--	225
313 to 265	Sedimentary Rock (Graywacke and Shale) Decomposed to Mod. Weathered, Mod. Soft to V. Soft	27 to 100	Clay	Stiff Clay (Without Water)	83	--	2,300	0.005	--

Notes: Elevations are based on project datum provided by Mark Thomas.
In soil layer, the buoyant unit weight should be used below design groundwater (elev. 305 feet)
For design scour consideration, no soil/rock support is available above the scour elevation.

SHEAR WAVE VELOCITY

A correlated shear wave velocity (V_{S30}) in the upper 30 meters (100 feet) of the soil profile of each boring completed for this project element (borings R-23-003 and R-23-004) 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 486 meters per second (about 1,594 ft/sec) is the average V_{S30} of the borings completed for this project element. This value corresponds to a “very dense soil and soft rock” with 360 m/s < Vs < 760 m/s for the upper 100 feet of the soil profile. The V_{S30} value was determined for this site based on the subsurface data obtained from the 2022 exploratory borings and correlations with SPT blow count N-values corrected for hammer efficiency using the equations outlined by Caltrans. For our evaluation, we used latitude 38.4879°N and longitude 122.4816° W for the site coordinates.

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

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[1] Caltrans Geotechnical Manual, Empirical Correlations for Estimating Shear Wave Velocity, January 2021.
[2] Caltrans Geotechnical Manual, Sampler Size Conversions to SPT N-value, Soil Correlations Module (March 2021).

Boring Data

R-23-003
North Abutment
NCRCD Sulphur Creek Fish Passage

Project: **NCRCD Sulphur Creek Fish Passage**
Job No.: **20-643.2**
Date: **1/30/23**
Boring: **R-23-003**
Support: **North Abutment**
By: **KBH**

SAMPLER TYPE

SPT	Do=2.0, Di=1.4
Standard CA	Do=2.5, Di=2.0
Modified CA	Do=3.0, Di=2.4

Boring Elevation: 314.8 feet
Groundwater Depth: 14.3 feet
Groundwater Elevation: 300.5 feet
Hammer Weight: 140.0 pounds
Hammer Drop: 30.0 inches
Hammer Efficiency (ER): 81.1 % (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	Depth to Bottom of	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	Moisture Content Wn	Total Unit Weight (pcf)	Average Mean Grain Size D_{50}	Percent Fines	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	2.000	1.400	10.0	15.0	304.8	GW	27	36				133.0	9.9	146.0											
2	3.000	2.000	15.0	20.0	299.8	GW	100	57				133.0	9.9	146.0		3									
3	2.000	1.400	20.0	25.0	294.8	GW	100	135				133.0	9.9	146.0											
4	2.000	1.400	25.0	30.0	289.8	CL	100	135				133.5	8.5	145.0											
5	2.000	1.400	30.0	35.0	284.8	CL	57	77				134.0	9.0	146.0											
6	2.000	1.400	35.0	40.0	279.8	CL	100	135				136.0	9.2	149.0											
7	2.000	1.400	40.0	45.0	274.8	CL	69	93				133.5	8.5	145.0											
8	2.000	1.400	45.0	50.0	269.8	CL	100	135				133.5	8.5	145.0											
9	2.000	1.400	50.0	55.0	264.8	CL	100	135				133.5	8.5	145.0											

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 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)

Empirical Correlations for Estimating Shear Wave Velocity, Caltrans Geotechnical Manual, January 2021

R-23-003
North Abutment
NCRCD Sulphur Creek Fish Passage

Project: **NCRCD Sulphur Creek Fish Passage**
Job No: **20-643.2**
Date: **1/30/23**
Boring: **R-23-003**
Support: **North Abutment**

Hammer Efficiency (ER): 81.1 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	**Sand**	**Clay/Silt**	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

Sample Number	**Depth (feet)**	**Depth to Bottom of Layer (feet)**	**Layer Thickness (feet)**	**Sample D_i (inches)**	**Soil Class.**	**Soil Type**	**Quaternary, Holocene or Pleistocene Enter Q, H or P ***	**Age Scaling Factor ASF (dim.)**	**Undrained Shear Strength S_u (psf)**	**Rock ****	**N ≤ 100 (bpf)**	**N_{SPT} ≤ 100 (bpf) *****	**N_{60} ≤ 100 (bpf)**	**N_{60} (bpf) (≤100)**	**σ'$_v$ (ksf)**	**σ'$_v$ (kPa)**	**d Layer Thickness in upper 30 m (m)**	**Layer Shear Wave Velocity, Vs SAND (m/sec)**	**GRAVEL (m/sec)**	**SILT/CLAY^1 (m/sec)**	**SILT/CLAY^2 (m/sec)**	**Sedimentary Rock (m/sec)**	**Soil/Rock Profile Profile Vs (m/sec)**	**Profile D/Vs (sec)**
1	10.0	15.0	15.0	1.4	GW	GRAVEL	P	NA			27	27	36	36	1.46	69.90	4.57		352				352	0.013
2	15.0	20.0	5.0	2.0	GW	GRAVEL	P	NA			100	42	57	57	2.15	102.77	1.52		399				399	0.004
3	20.0	25.0	5.0	1.4	GW	GRAVEL	P	NA			100	100	100	100	2.56	122.78	1.52		448				448	0.003
4	25.0	30.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	2.98	142.79	1.52					474	474	0.003
5	30.0	35.0	5.0	1.4	CL	CLAY	Q	1.00		rock	57	57	77	77	3.40	162.57	1.52					436	436	0.003
6	35.0	40.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	3.81	182.58	1.52					474	474	0.003
7	40.0	45.0	5.0	1.4	CL	CLAY	Q	1.00		rock	69	69	93	93	4.25	203.31	1.52					463	463	0.003
8	45.0	50.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	4.66	223.09	1.52					474	474	0.003
9	50.0	55.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	5.07	242.86	1.52					474	474	0.003
																			Sum =	**16.76**							**444**	**0.040**

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** 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)	16.76
Vs(d) (m/sec)	420
V_{S30} (m/sec)	**504**
V_{S30} (ft/sec)	**1653**

Soil Profile Type
'C' (360 m/s < Vs < 760 m/s)

Shear Wave Velocity Correlations (valid for 3 ≤ N_{60} ≤ 100)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ for Pleistocene
Clay^1: $Vs = 203(S_u/P_a)^{0.475}$
Clay^2: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$
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) based on SPT correlations are valid where 3 ≤ N_{60} ≤ 100.
4) Undrained Shear Strength (S_u) based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane; or 0.5(Pocket Penetrometer) in psf.

Boring Data

R-23-004
South Abutment
NCRCD Sulphur Creek Fish Passage

Project: **NCRCD Sulphur Creek Fish Passage**
Job No.: **20-643.2**
Date: **1/30/23**
Boring: **R-23-004**
Support: **South Abutment**
By: **KBH**

SAMPLER TYPE
SPT Do=2.0, Di=1.4
Standard CA Do=2.5, Di=2.0
Modified CA Do=3.0, Di=2.4

Boring Elevation: 319.1 feet
Groundwater Depth: 18.6 feet
Groundwater Elevation: 300.5 feet
Hammer Weight: 140.0 pounds
Hammer Drop: 30.0 inches
Hammer Efficiency (ER): 81.1 % (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	Depth to Bottom of	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	Moisture Content Wn	Total Unit Weight (pcf)	Average Mean Grain Size D_{50}	Percent Fines	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	2.000	1.400	5.0	10.0	314.1	GC	47	64				133.0	10.0	146.0											
2	2.000	1.400	10.0	20.0	309.1	CL	100	135				134.0	9.0	146.0											
3	2.000	1.400	20.0	25.0	299.1	CL	45	61				134.0	9.0	146.0											
4	2.000	1.400	25.0	30.0	294.1	CL	65	88				134.0	9.0	146.0											
5	2.000	1.400	30.0	35.0	289.1	CL	100	135				134.0	9.0	146.0											
6	2.000	1.400	35.0	40.0	284.1	CL	27	36				134.0	9.0	146.0											
7	3.000	2.400	40.0	45.0	279.1	CL	58	51	4.50			134.0	9.0	146.0											
8	3.000	2.400	45.0	50.0	274.1	CL	100	88				134.0	9.0	146.0											
9	3.000	1.400	50.0	55.0	269.1	CL	49	20	4.50			134.0	9.0	146.0											
10	2.000	1.400	55.0	60.0	264.1	CL	100	135				134.0	9.0	146.0											
11	2.000	1.400	60.0	62.0	259.1	CL	100	135				134.0	9.0	146.0											
12	2.000	1.400	62.0	65.0	257.1	CL	100	135				134.0	9.0	146.0											

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 for the Soil Type.
NP = Non Plastic
NA = Not Applicable
SPT = Standard Penetration Test
Liquidity Index (LI) = (Wn-PL)/(LL-PL)

Shear Wave Velocity (Vs)

Empirical Correlations for Estimating Shear Wave Velocity, Caltrans Geotechnical Manual, January 2021

R-23-004
South Abutment
NCRCD Sulphur Creek Fish Passage

Project: **NCRCD Sulphur Creek Fish Passage**
Job No: **20-643.2**
Date: **1/30/23**
Boring: **R-23-004**
Support: **South Abutment**

Hammer Efficiency (ER): 81.1 %

INPUT	CALCULATION

Dimensionless Age Scaling Factor (ASF)			
Geologic Time	**Sand**	**Clay/Silt**	
Q = Quaternary	1.00	1.00	last 2.6 million years
H = Holocene	0.90	0.88	last 11,700 years
P = Pleistocene	1.17	1.12	from 11,700 years to 2.6 million years

																		Layer Shear Wave Velocity, Vs					**Soil/Rock Profile**	
Sample Number	**Depth (feet)**	**Depth to Bottom of Layer (feet)**	**Layer Thickness (feet)**	**Sample D_i (inches)**	**Soil Class.**	**Soil Type**	**Quaternary, Holocene or Pleistocene** **Enter Q, H or P ***	**Age Scaling Factor ASF (dim.)**	**Undrained Shear Strength S_u (psf)**	**Rock ****	**N ≤ 100 (bpf)**	**N_{SPT} ≤ 100 (bpf) *****	**N_{60} ≤ 100 (bpf)**	**N_{60} (bpf) (≤100)**	**σ'$_v$ (ksf)**	**σ'$_v$ (kPa)**	**d Layer Thickness in upper 30 m (m)**	**SAND (m/sec)**	**GRAVEL (m/sec)**	**SILT/CLAY^1 (m/sec)**	**SILT/CLAY^2 (m/sec)**	**Sedimentary Rock (m/sec)**	**Profile Vs (m/sec)**	**Profile D/Vs (sec)**
1	5.0	10.0	10.0	1.4	GC	GRAVEL	P	NA			47	47	64	64	0.73	34.95	3.05		357				357	0.009
2	10.0	20.0	10.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	1.46	69.90	3.05					474	474	0.006
3	20.0	25.0	5.0	1.4	CL	CLAY	Q	1.00		rock	45	45	61	61	2.83	135.63	1.52					405	405	0.004
4	25.0	30.0	5.0	1.4	CL	CLAY	Q	1.00		rock	65	65	88	88	3.25	155.64	1.52					455	455	0.003
5	30.0	35.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	3.67	175.65	1.52					474	474	0.003
6	35.0	40.0	5.0	1.4	CL	CLAY	Q	1.00		rock	27	27	36	36	4.09	195.67	1.52					342	342	0.004
7	40.0	45.0	5.0	2.4	CL	CLAY	Q	1.00	4500	rock	58	38	51	51	4.50	215.68	1.52					382	382	0.004
8	45.0	50.0	5.0	2.4	CL	CLAY	Q	1.00		rock	100	65	88	88	4.92	235.70	1.52					455	455	0.003
9	50.0	55.0	5.0	1.4	CL	CLAY	Q	1.00	4500	rock	49	15	20	20	5.34	255.71	1.52					283	283	0.005
10	55.0	60.0	5.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	5.76	275.72	1.52					474	474	0.003
11	60.0	62.0	2.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	6.18	295.74	0.61					474	474	0.001
12	62.0	65.0	3.0	1.4	CL	CLAY	Q	1.00		rock	100	100	100	100	6.34	303.74	0.91					474	474	0.002
																				Sum =	**19.81**						**421**	**0.049**

* For SAND, CLAY and SILT enter Q, H or P; For GRAVEL enter H or P
** 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)	19.81
Vs(d) (m/sec)	405
V_{S30} (m/sec)	**467**
V_{S30} (ft/sec)	**1532**

Soil Profile Type
'C' (360 m/s < Vs < 760 m/s)

Shear Wave Velocity Correlations (valid for $3 \le N_{60} \le 100$)

Sand: $Vs = 30(ASF)(N_{60})^{0.23}(\sigma'_{vo})^{0.23}$
Silt: The SPT N_{60} correlation recommended for cohesive soil layers is also recommended for silt layers.
Gravel: $Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$ for Holocene
Gravel: $Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$ for Pleistocene
Clay^1: $Vs = 203(S_u/P_a)^{0.475}$
Clay^2: $Vs = 26(ASF)(N_{60})^{0.17}(\sigma'_{vo})^{0.32}$
Young Sedimentary Rock (Tertiary Deposits): $Vs = 109(N_{60})^{0.319} \le 560$/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) based on SPT correlations are valid where $3 \le N_{60} \le 100$.
4) Undrained Shear Strength (S_u) based on 0.5(UCS); or in-situ Vane Shear; or in-situ Torvane; or 0.5(Pocket Penetrometer) in psf.

DEEP FOUNDATIONS

Recommendations are summarized below for cast-in-drilled-hole (CIDH) pile foundations at abutments. Refer to Section 11 of the Foundation Report that summarizes the foundation data and loading conditions provided by MT that were used in our pile analysis.

COMPRESSIVE RESISTANCE

The side (compressive) resistance for the CIDH pile foundations was evaluated using Load and Resistance Factor Design (LRFD) method and factors from AASHTO LRFD Bridge Design Specifications (BDS), 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 BDS 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 BDS were used for cohesionless (gravel) layers.

The idealized geotechnical parameters shown in Tables V-1 were used to calculate the design tip elevations for 30-inch diameter CIDH piles at both abutments. Design groundwater was modeled at elev. 305 feet. A total scour elevation of 302.4 feet was provided by WRA.

Skin friction contributions are only considered in our compressive resistance analysis. For our foundation design analysis, the top 5 feet of the pile or the depth of scour below the pile cap (whichever is lower) is excluded from contributing to geotechnical capacity. Tip resistance in axial compression was neglected in consideration of slurry installation method, consistent with current Caltrans guidelines for CIDH pile design.

A geotechnical resistance factor (φ_{qs}) of 0.7 for skin friction was used to determine the compressive resistance at the Strength Limit State consistent Caltrans amendments to AASHTO LRFD method for the CIDH piles.

Refer to the 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 CIDH piles.

TENSION (UPLIFT) RESISTANCE

No tension demands are indicated for the CIDH pile foundations at this time.

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

NCRCD Sulphur Creek Fish Passage

St Helena

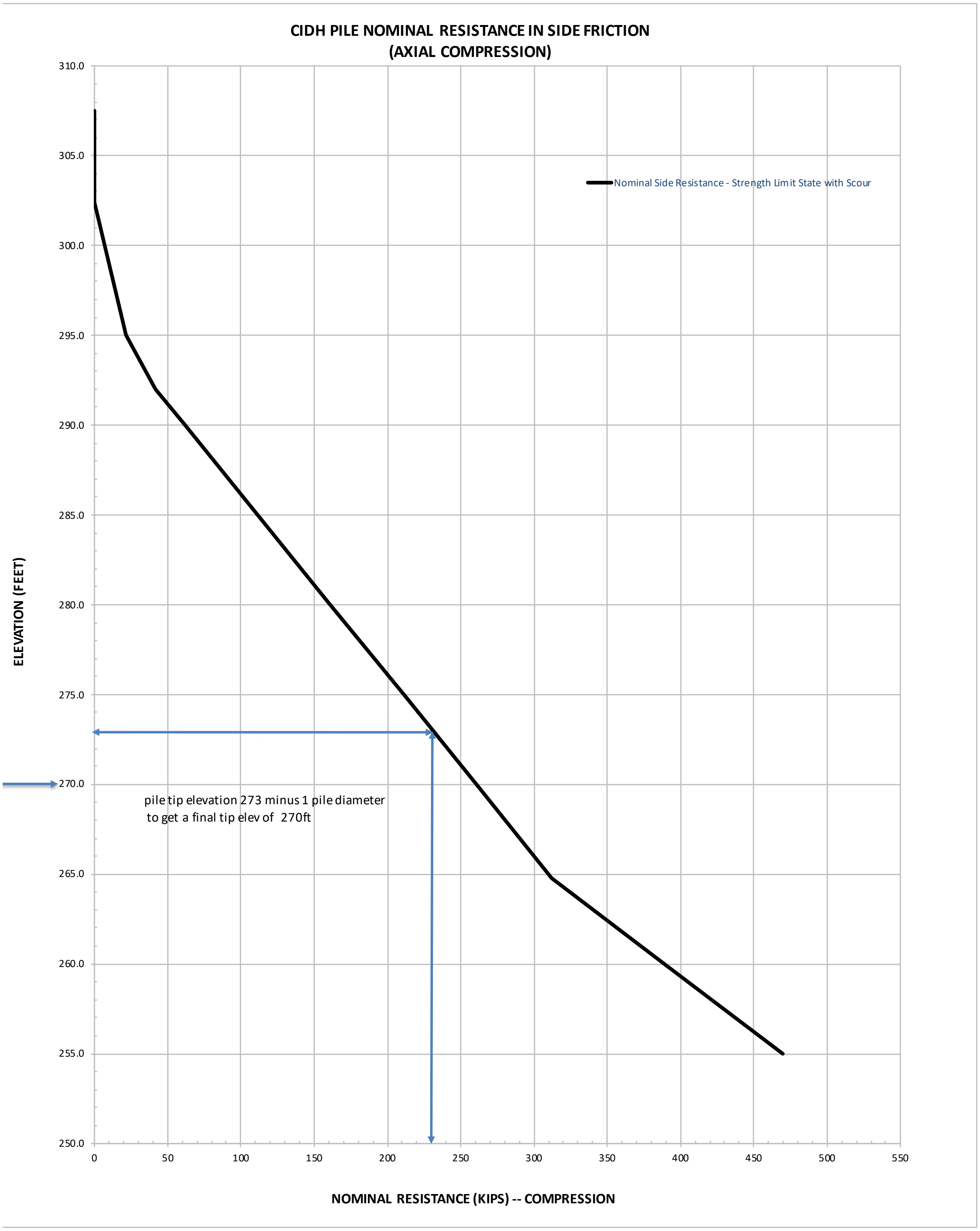
Crawford Project Number: 20-643.1

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

July 23, 2024

Support Location(s): Abutment 1 (North)	**Pile Diameter = 30 inches**	**Pile Cut-Off Elevation = 307.5 feet**
Boring(s): R-23-003	**Socket Diameter = NA**	**Permanent Casing Tip Elevation = NA**

SERVICE LIMIT			**STRENGTH LIMIT**			**EXTREME LIMIT**		
REQUIRED NOMINAL RESISTANCE =	110	kips	REQUIRED NOMINAL RESISTANCE =	230	kips	REQUIRED NOMINAL RESISTANCE =	NA	kips
SCOUR ELEVATION =	302.4	feet	SCOUR ELEVATION =	302.4	feet	SCOUR ELEVATION =	302.4	feet
DESIGN PILE TIP ELEVATION =	NA	feet	DESIGN PILE TIP ELEVATION =	270.0	feet	DESIGN PILE TIP ELEVATION =	NA	feet



CIDH PILE NOMINAL RESISTANCE IN SIDE FRICTION
(AXIAL COMPRESSION)

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 (COMPRESSION)

NCRCD Sulphur Creek Fish Passage

St Helena

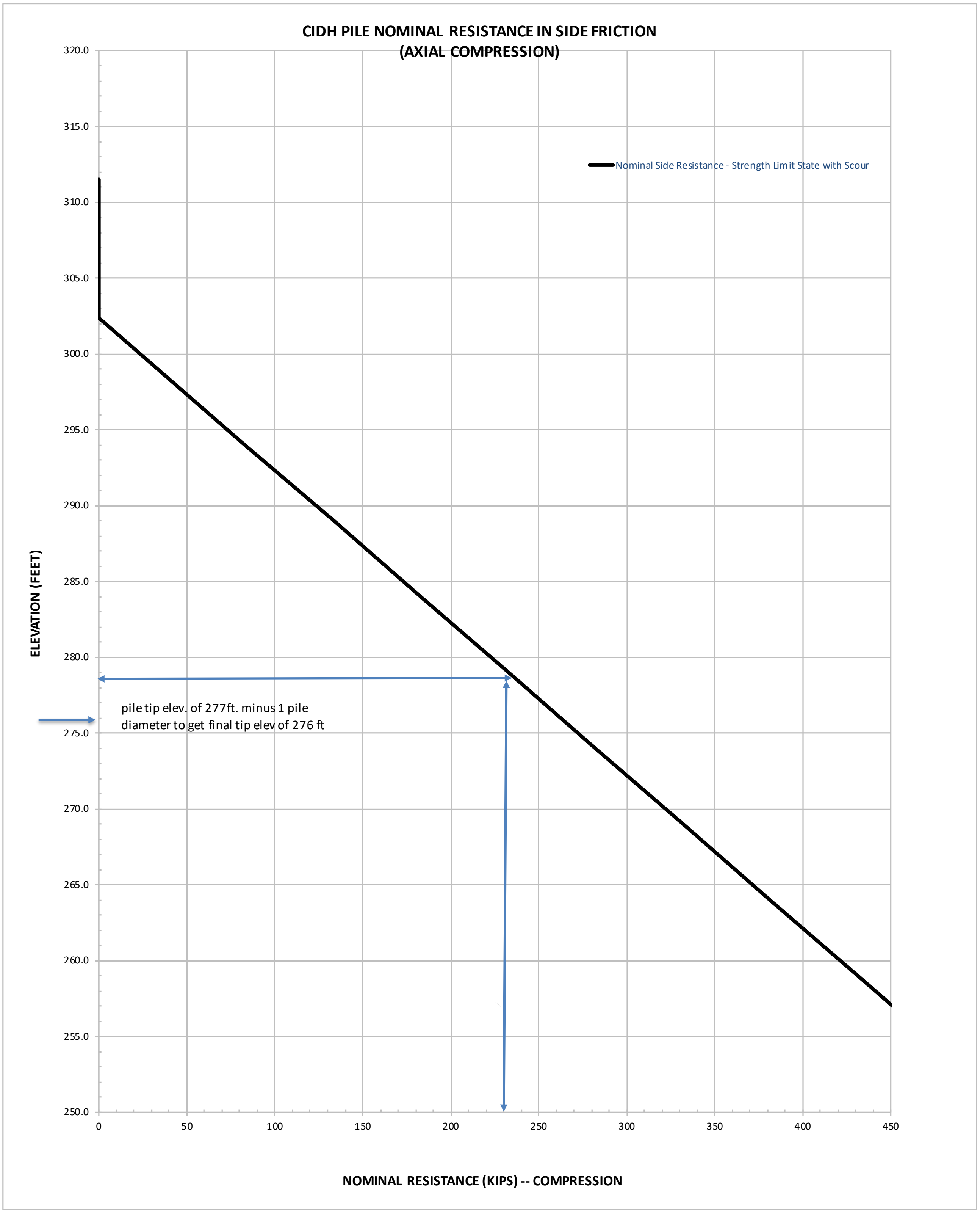
Crawford Project Number: 20-643.1

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

July 24, 2024

Support Location(s): Abutment 2 (South)	**Pile Diameter = 30 inches**	**Pile Cut-Off Elevation = 311.5 feet**
Boring(s): R-23-004	**Socket Diameter = NA**	**Permanent Casing Tip Elevation = NA**

SERVICE LIMIT			**STRENGTH LIMIT**			**EXTREME LIMIT**		
REQUIRED NOMINAL RESISTANCE =	110	kips	REQUIRED NOMINAL RESISTANCE =	230	kips	REQUIRED NOMINAL RESISTANCE =	NA	kips
SCOUR ELEVATION =	302.4	feet	SCOUR ELEVATION =	302.4	feet	SCOUR ELEVATION =	302.4	feet
DESIGN PILE TIP ELEVATION =	NA	feet	DESIGN PILE TIP ELEVATION =	276.0	feet	DESIGN PILE TIP ELEVATION =	NA	feet



pile tip elev. of 277ft. minus 1 pile diameter to get final tip elev of 276 ft

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

LATERAL RESISTANCE

We used LPile Version 2018.10.09 software to evaluate lateral pile capacity to evaluate the 30 in CIDH piles at the abutments.

For the proposed bridge, pile response in the longitudinal and transverse bridge directions was computed with an axial compression equal to the Service State Load per Pile which gets applied to the top of the pile (85.26 kips). Crawford determined the allowable lateral pile design loads that would produce approximately ¼-in displacement (Tables V-4 and V-5). All lateral displacement was analyzed using a pinned (free-head) condition. The geotechnical factor of one ($\varphi = 1.0$) was used in our lateral load analysis. The pile spacing from foundation plan in the 65% plans (received April 13, 2023) is summarized in Table V-3.

Table V-3: Support Pile Spacing

Support	Longitudinal Pile Spacing	Transverse Pile Spacing*
Abutment 1	6.25 ft	6.25 ft
Abutment 2	6.25 ft	6.25 ft

We show our LPile lateral pile analysis results, which includes the p-multipliers factors consistent with Table 10.7.2.4-1 of the *California Amendments to AASHTO BDS*., in Tables V-4 and V-5. The LPile input data and output graphs for the lowest p-multiplier are included in this Appendix.

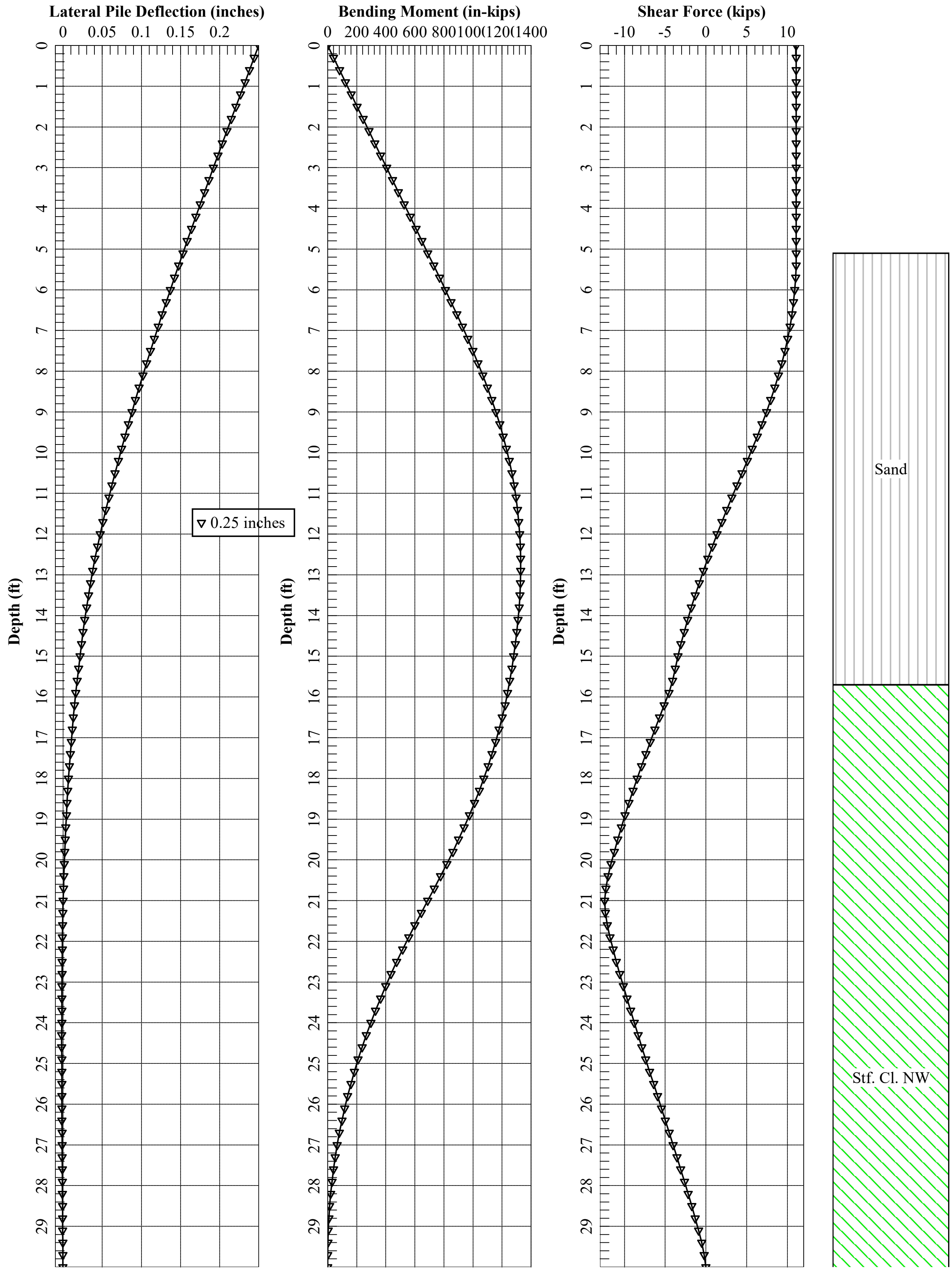
Table V-4: Abutment 1 Pile Head Deflection vs. Lateral Load

Condition	Pile Row	P-multiplier	Pile Head Deflection (inches)	Allowable Shear Force (kips)
Longitudinal (2.5B)	Row 1	0.80	0.25	13
Transverse (2.5B)	Row 1	0.68	0.25	13
	Row 2	0.45	0.25	11
	Row 3+	0.33	0.25	11

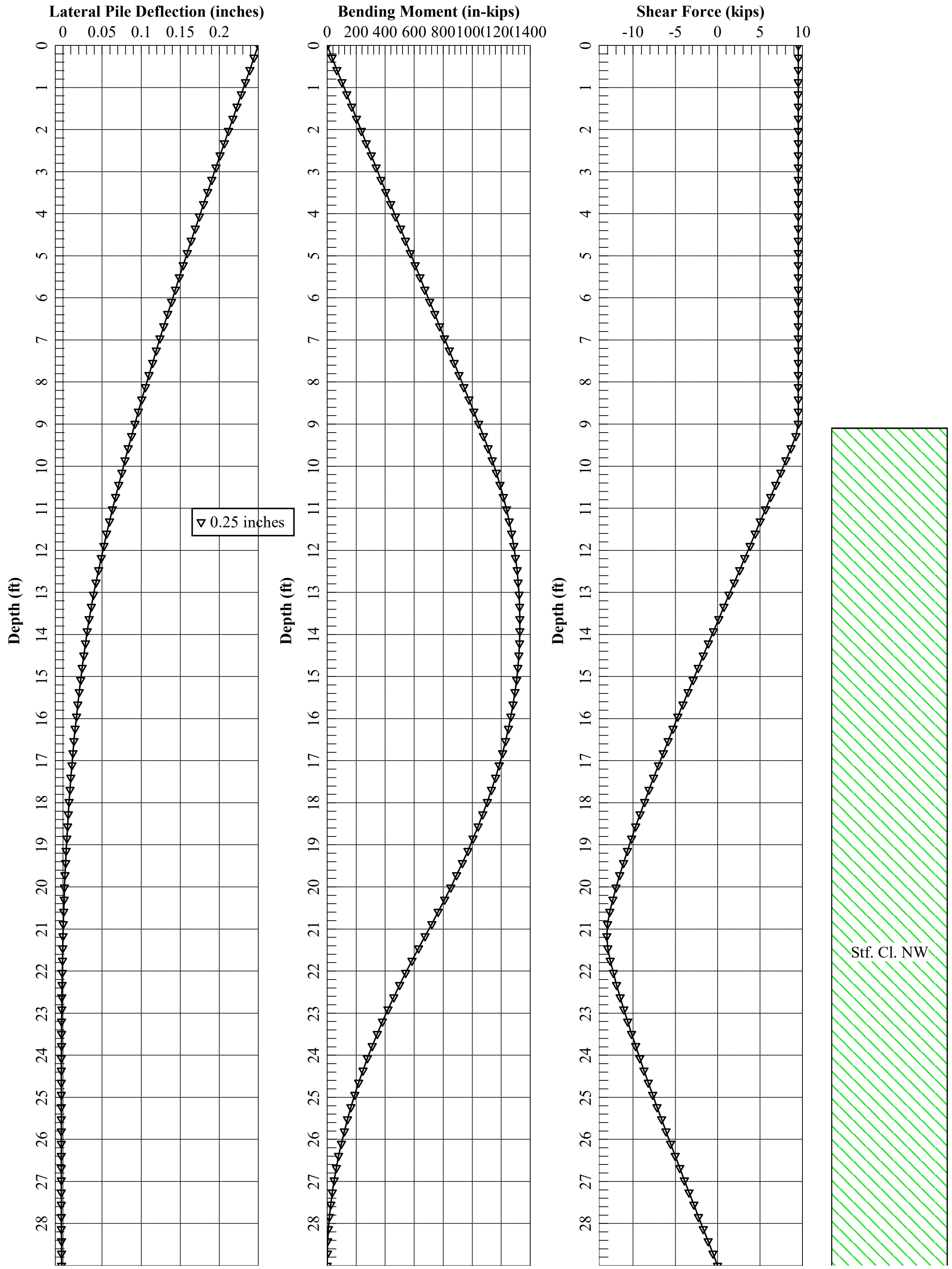
Table V-5: Abutment 2 Pile Head Deflection vs. Lateral Load

Condition	Pile Row	P-multiplier	Pile Head Deflection (inches)	Allowable Shear Force (kips)
Longitudinal (2.5B)	Row 1	0.80	0.25	12
Transverse (2.5B)	Row 1	0.68	0.25	12
	Row 2	0.45	0.25	11
	Row 3+	0.33	0.25	10

Sulphur Creek - Abutment 1 - 30" CIDH - Modeled From 311.5' - P-Mult = 0.33



Sulphur Creek - Abutment 2 - 30" CIDH - Modeled From 307.5' - P-Mult = 0.33



NEGATIVE SKIN FRICTION

Negative skin friction is not indicated/expected to develop for the pile foundations at this site and is not a design consideration for this project.

SETTLEMENT

Based on the subsurface data obtained for this study, total settlement at each support under service load is estimated to be within the permissible 1.0-inch settlement for the recommended pile foundations. Since the piles will be embedded adequately into soft rock, and the piles will not be subjected to downdrag loads, a detailed assessment of the pile group settlement is not considered warranted.

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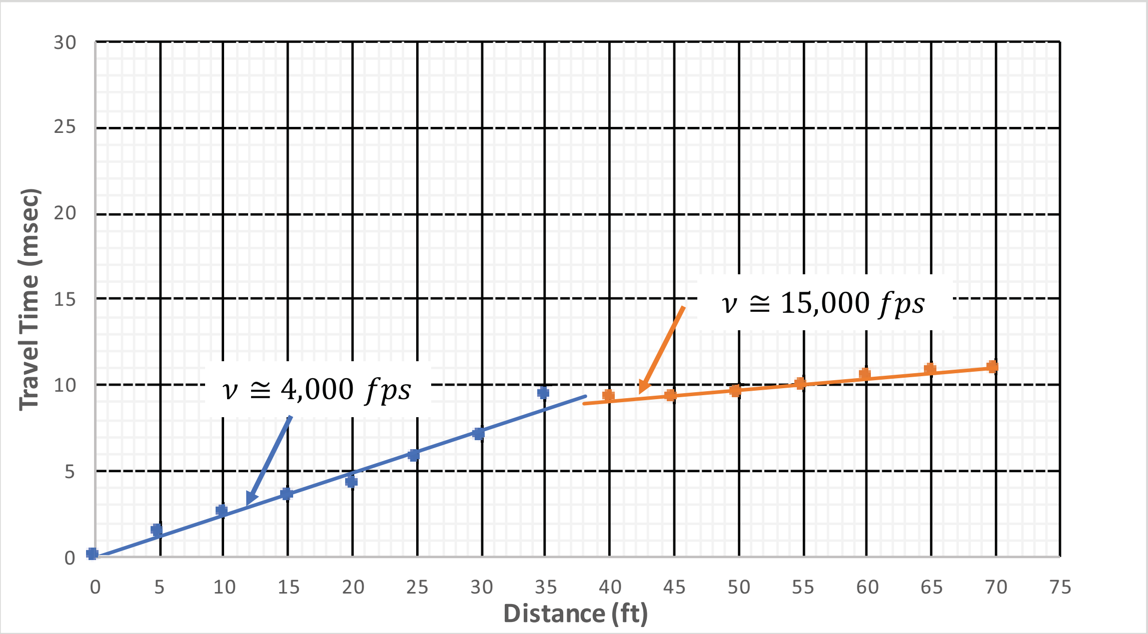
Taber
Since 1954

Project Name: NCRCD - Sulphur Creek Fish Passage
Project Number: 20-643.1
Date: February 10, 2021
Location: St. Helena, CA



Seismic Refraction Data Log

Line Number: S3	Line Length: 100-ft.	Shot Location: 0-ft.	Orientation: 238°SW

Approximate Depth to Rock: 13-ft.

Geophone Number	Impact Distance (ft)	Travel Time (msec)
14	0	0.0
13	5	1.5
12	10	2.6
11	15	3.6
10	20	4.3
9	**25**	5.8
8	**30**	7
7	**35**	9.4
6	**40**	9.2
5	**45**	9.3
4	**50**	9.5
3	**55**	10
2	**60**	10.5
1	**65**	10.8
0	**70**	10.9