Appendix C – Preliminary Foundation Report

PRELIMINARY FOUNDATION REPORT
ARROYO ROAD OVER DRY CREEK BRIDGE REPLACEMENT
ALAMEDA COUNTY, CALIFORNIA

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WRECO Project No. P19070
June 2021

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WRECO

June 11, 2021

Wood Rodgers. Inc.
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Attention: Chris Hodge, PE
Project Manager

Subject: Preliminary Foundation Report
Arroyo Road Over Dry Creek Bridge Replacement
Alameda County, California
WRECO Project No. 19070

WRECO is pleased to submit our *Preliminary Foundation Report for the Arroyo Road over Dry Creek Bridge Replacement*. This report was conducted and prepared in general conformance with the scope of work prepared by WRECO for the subject project.

We would like to thank Wood Rodgers. Inc. and Alameda County for the opportunity to prepare this Foundation Report.

If you have any questions or wish to discuss this report in greater detail, please contact me at (916) 515-7428.

Sincerely,
WRECO

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PRELIMARY FOUNDATION REPORT
ARROYO ROAD OVER DRY CREEK BRIDGE REPLACEMENT
ALAMEDA COUNTY, CALIFORNIA

1. INTRODUCTION

Alameda County Public Works Agency is proposing to replace the structurally deficient Arroyo Road over Dry Creek Bridge (33C0448) with a new bridge that meets current applicable County, American Association of State Highway and Transportation Officials (AASHTO), and Caltrans design criteria and standards. In addition to the new bridge, the proposed project will ensure the roadway through the project limits meets current County and AASHTO standards and will provide a Class I bike path over the bridge. The project is funded primarily through the state set-aside of Federal funds for the Highway Bridge Program (HBP), as administered through Caltrans Local Assistance. The Class I Bike Path will be funded using local dollars.

1.1. Scope of Work

WRECO's scope of work for the proposed Project consisted of the following:

- Visited the site and marked out in white paint the proposed boring locations and called Underground Service Alert (USA) North Dig Alert a minimum of 72-hours prior to the start of the field investigation work to identify potential underground utility conflicts.
- Paid all fees and obtained a well construction/destruction (boring) permit from the County Department of Environmental Health to perform borings at the Project site.
- Obtained an encroachment permit to perform work within the City of Livermore's right-of-way (at no cost to WRECO).
- Performed a literature search for readily available published geologic and geohazards information at and in the near vicinity of the Project site.
- Drilled two (2) soil borings to completion depths of approximately 58 and 70 feet below existing road grade.
- Tested representative soil samples in the laboratory to better determine their engineering properties. Laboratory testing consisted of Atterberg Limits, grain size distribution, soil corrosive potential, and Unconfined Compression testing.

WRECO prepared a *Preliminary Foundation Report* that provides design and construction recommendations for the bridge replacement. The report includes the following:

- A description of the geotechnical work performed.
- A Project summary and description of the proposed improvements.
- A brief overview of the field investigation performed as part of this study.
- A summary of the laboratory testing performed as part of this study.
- A discussion of the regional and site geology as it pertains to the proposed improvements.
- A preliminary discussion of the regional seismology and assumed preliminary seismic design parameters for the proposed Project site in accordance with the Caltrans ARS Online Design Tool V 3.0.2 and the *Caltrans Seismic Design Criteria, Version 2.0, April 2019.*

- A liquefaction analysis and the predicted seismic displacements anticipated to occur at the Project site when subjected to the design seismic event.
- A summary and discussion of the available as-built information as it pertains to the proposed foundation selection.
- A discussion of the preliminary foundation recommendations for the proposed bridges taking into account the preliminary loading demands, site soil conditions, and environmental constraints.

1.2. Project Description

Alameda County is proposing to replace the Arroyo Road Bridge over Dry Creek (existing Bridge No. 33C0448). The primary objective of the Project is to replace the 90-year-old bridge structure over Dry Creek. It is proposed to replace the bridge on the existing alignment.

1.3. Project Location

The existing bridge is located on Arroyo Road, approximately 1.87 miles east of State Route (SR) 84, and 0.87 miles southeast of the City of Livermore limits. The Project site is a bridge over Dry Creek on Arroyo Road with a wooden pedestrian passage bridge on the northeast side. Arroyo Road is a two-lane road generally oriented north to south, but is oriented roughly southwest to northeast at the bridge locations.

The existing bridge crosses Dry Creek, which runs generally from northeast to southwest in the Project vicinity, taking a turn to the south just to the west of the existing bridge where it runs against the base of the hill located to the northwest of the bridge. To the south and southeast of the bridge the land is generally level, with low hills bordering the Project to the east. The Project is surrounded by agricultural land to the north and west with a public park to the south and a golf course and reservoir to the east.

The Project location and site features are shown on the attached Vicinity Map, Figure 1 (See Appendix I for Site Maps). The approximate location of the Project is 37.63781° N, 121.76364° W. Elevations in this report are referenced to North American Vertical Datum 88 (NAVD 88).

1.4. Existing and Proposed Bridges

The existing structure consists of single-span bridge with a deck composed of concrete encased steel girders supported on concrete seat type abutments. The foundation type is listed as unknown (BIRIS, 2015), but is assumed to be spread footings. The existing bridge is approximately 24 feet (ft) long and 23 ft wide, and was originally constructed in 1930, and then reconstructed in 1937 with the addition of curbs.

The guardrail on the western side of the bridge is missing and has been replaced with a chain link fence. The concrete in the deck and beams has extensive spalling and exposed rebar and the beams are corroded. There is reported section loss of the web and flanges due to corrosion.

The County proposes to replace the existing bridge with a cast-in-place, reinforced concrete, single-span slab bridge that will accommodate two travel lanes plus shoulders and traffic rated vehicular barriers to meet AASHTO standards. The bridge will also accommodate a 12-foot-wide Class I bike path separated from traffic by an interior vehicular traffic rated barrier. The replacement structure will be 34-feet-long and will be supported by integral diaphragm type abutments on deep foundations.

The roadway profile will be raised approximately two feet to meet hydraulic and geometric requirements. To accommodate the raised profile, wider bridge structure, and longer span, the roadway centerline at the bridge will be shifted to the southwest to maintain traffic throughout construction while balancing impacts from slopes encroaching upon agricultural land (winery) to the northwest, a park to the southwest, grazing land to the northeast, and a recreational facility to the southeast.

The access driveway will be reconstructed to connect into the raised roadway.

Overhead electric lines on wooden poles run along the south side of the roadway, and overhead telecommunication lines on wooden poles run along the north side of the roadway. There is an abandoned underground waterline along the north side of the roadway, crossing the creek via attachment to the existing bridge. Additional private potable and irrigation water lines run along the north side of the roadway within the private frontage road with service drop lines running easterly. To accommodate the widened roadway, the proposed Project includes the following:

- Overhead utility lines and support poles along both the north and south side will require permanent relocation.
- Abandoned water line will be removed with the existing bridge within the limits of excavation for the new bridge and capped within the approach roadway.
- No modifications are expected to the private water lines.

2. EXCEPTIONS TO POLICY

No exceptions to policy were taken in preparation of this *Preliminary Foundation Report*.

3. FIELD INVESTIGATION AND TESTING PROGRAM

The subsurface conditions in the vicinity of the proposed bridge abutments were characterized by means of five short auger borings designated A-20-001 through A-20-005 and two deep rotary borings designated R-20-006 and R-20-007. Borings A-20-001 through A-20-005 were made in the roadway, but encountered unanticipated concrete pavement under approximately 0.5 ft of asphalt concrete pavement. After several attempts to penetrate the pavement subsequent borings were moved off of the roadway to locations away from overhead and underground utilities to allow reaching planned depths of exploration. The borings were drilled by Geo-Ex Drilling on January 29, 2020.

The borings were used to obtain disturbed and relatively undisturbed representative soil samples to characterize the soil conditions at the proposed foundation locations. The recovered soil samples were logged by an on-site WRECO engineer as drilling progressed using the procedures in the *2015 Caltrans Soil and Rock Logging, Classification, and Presentation Manual*.

The soil samplers were advanced/driven using a 140-pound auto-trip hammer, free falling 30-inches, in accordance with Standard Penetration Test (SPT) (ASTM D1586) procedures. Field blow counts were recorded as the number of hammer blows required to drive the sampler through the final 12 inches of an 18-inch drive or refusal. The recorded blow counts at specific depths are shown on the Log of Test Borings (LOTB) in Appendix III.

A summary table of the boring locations, ground surface elevations, drilled depths, and hammer efficiency ratios are provided in the following table.

Table 1. Summary of Boring Information

Boring ID	Completion Date	Drill Rig Type	Hammer Type	Hammer Efficiency Ratio (%)	Approximate Surface Elevation (feet)	Drilled Depth (feet)
A-20-001	Jan. 29,2020	CME-75	Automatic	77	512.8	0.5±
A-20-002	Jan. 29,2020	CME-75	Automatic	77	512.6	0.5±
A-20-003	Jan. 29,2020	CME-75	Automatic	77	512.4	0.5±
A-20-004	Jan. 29,2020	CME-75	Automatic	77	512.3	0.5±
A-20-005	Jan. 29,2020	CME-75	Automatic	77	512.2	0.5±
R-20-006	Jan. 29,2020	CME-75	Automatic	77	509.3	58.0
R-20-007	Jan. 29,2020	CME-75	Automatic	77	512.9	70.0
Notes: CME = Central Mine Equipment R-20-001 encountered concrete below the roadway and was moved until boring could be completed.						

Detailed visual descriptions of the recovered soil samples from the borings are presented on the LOTB in Appendix III.

4. LABORATORY TESTING PROGRAM

Laboratory soil testing for this study consisted of grain size determination, Atterberg Limits, corrosive potential (i.e. sulfate content, pH, resistivity, and chloride content testing), and R-value testing. A summary of the laboratory testing is provided in Table 2.

Table 2. Laboratory Test Summary

Boring ID	Sample Depth/Interval (ft)	Test	Standard (ASTM/CTM)
R-20-006	0.0 – 5.0	R-Value	CTM 301
	20 – 21.5	Atterberg Limits, Grain Size Distribution	ASTM D4318, ASTM D6913
	38.0 – 43.0	Atterberg Limits, Grain Size Distribution, Unconfined Compression, Corrosivity	ASTM D4318, ASTM D6913, ASTM D2166, CTM 422
	51.5 – 56.5	Atterberg Limits, Grain Size Distribution, Unconfined Compression	ASTM D4318, ASTM D6913, ASTM D2166
R-20-007	10.0 – 11.5	Grain Size Distribution	ASTM D6913
	40.0 – 41.5	Atterberg Limits, Grain Size Distribution	ASTM D4318, ASTM D6913

Notes:
ASTM: American Society for Testing and Materials
CTM: California Test Method

The samples tested are shown on the LOTB, which is included in Appendix III. Copies of the laboratory test results can be found in Appendix IV. Results of the corrosive potential to buried steel and concrete testing are further discussed in Section 9 of this report.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1. Geology

The Project is located within the Coast Ranges Geomorphic Province of California. This province extends along most of the California coast and is bounded by the Great Valley and Klamath Mountains to the east, the Pacific Ocean to the west, the Transverse Range Mountains Ranges to the south and the California-Oregon border to the north. Much of the Coast Range province is composed of marine sedimentary deposits and volcanic rocks that form northwest trending mountain ridges and valleys, running subparallel to the San Andreas Fault Zone. The relatively thick marine sediments dip east beneath the alluvium of the Great Valley.

The Coast Ranges can be further divided into the northern and southern ranges, which are separated by the San Francisco Bay. The San Francisco Bay lies within a broad depression created from an east-west expansion between the San Andreas and the Hayward fault systems.

The existing bridge and the southern approach are mapped as underlain by Quaternary aged Surficial Sediments (Qg and Qa). The Surficial Sediments consist of alluvial gravel, sand, and clay of valley areas along both banks and to the southeast of bridge and sand and gravel of major stream channels along the channel.

Older Surficial Sediments (Qoa[2]) are shown underlying the northern approach and are composed of older alluvial gravel and sand. The Livermore gravel (QTIg) is shown underlying the hills to the east and is described as cobble-pebble gravel and sand. Pliocene-aged Orinda Formation

(Tor) is shown along the base of the hills to the southeast of the bridge and is described as pebble conglomerate, sandstone, claystone, interbedded, conglomerate of mostly Franciscan detritus.

The Los Positas Fault trace is shown crossing below or close to the bridge and is shown as obscured (inferred trace obscured by sediment) near the bridge. The fault trace generally matches the base of the hills that run to the northeast.

The soils and rock observed at the site generally match those shown on the geologic mapping, except the bedded sedimentary rock was observed along the eroded base of the hill immediately north and downstream of the bridge where the channel has eroded the base of the hill. The rock was exposed from near channel grade approximately 10 ft downstream of the bridge increasing in elevation to the west. This rock appears to match the description of the Orinda Formation and consisted of intensely weathered claystone dipping approximately 20 degrees to the north. Above the weathered rock, approximately 10 to 15 ft of gravel with sand was observed, which appears to be alluvium.

The Project site is shown in relation to the published geology on Figure 2, Geologic Map, in Appendix I.

5.2. Subsurface Conditions

Based on the two exploratory borings performed by WRECO, subsurface conditions at the site generally consist of a variable thickness of fill and alluvium over decomposed rock with less weathered rock at depth. Boring made for the separate environmental study encountered Portland cement concrete below the asphalt concrete pavement surface. The soils have been spilt into three general engineering units including alluvial soils, decomposed rock and intensely weathered rock.

<u>Unit 1 – Fill and Alluvium</u>

The upper-most unit consists of a thin layer of gravelly silt with sand, which can be attributed to alluvial fill and top soil. Along the roadway, approximately 6 inches of asphalt concrete (AC) was encountered above Portland cement concrete (PCC) pavement. The drill rig was incapable of penetrating the PCC and thickness was not determined.

<u>Unit 2 – Decomposed Rock to Unit 3 - Intensely Weathered Rock</u>

This unit is composed of decomposed rock that grades to intensely weathered rock. The decomposed rock is soil like and consists of dense to very dense clayey sand with gravel and medium stiff silty clay with gravel which extends to approximately elevation 473- 483 feet. The rock grades to intensely weathered rock below these elevations and consists of very hard clay to soft claystone encountered to the maximum depth explored (elevation 441.9 feet). This rock was observed along the northern bank downstream of the bridge and appears to be at the surface or at shallow depth within the footprint of the proposed northern abutment. The exposed rock was intensely weathered to decomposed.

The earth materials encountered in the borings are summarized in Table 3.

Table 3. Subsurface Conditions Summary

Support	Abutment 1	Abutment 2
Boring	**R-20-001**	**R-20-002**
"A" line Stationing	11.6' Rt of STA 15+14.5	75.7' Lt of STA 13+02.1
top hole elev.	509.3	512.9
Unit 1	Gravelly Silt with Sand (5±ft fill?) γ (pcf) 120, 65 bouyant φ (°) 40 c (psf) 0	Gravelly Silt with Sand (5±ft fill?) γ (pcf) 120, 65 bouyant φ (°) 40 c (psf) 0
elev.	495	483
Unit 2	Clayey Sand with Gravel – Dense to Very Dense (Decomposed Claystone) γ (pcf) 120, 65 bouyant φ (°) 40 c (psf) 0	Silty Clay with Gravel – Hard (Decomposed Claystone) γ (pcf) 120, 65 bouyant φ (°) 40 c (psf) 0
elev.	483	473
Unit 3	Lean Clay – Very Hard (Intensely Weathered Claystone) γ (pcf) 120, 65 bouyant φ (°) 0 c (psf) 4000	Lean Clay – Very Hard (Intensely Weathered Claystone) γ (pcf) 120, 65 bouyant φ (°) 0 c (psf) 4000
Bottom hole elev.	452.3	441.9

For the boring locations and the actual descriptions of the soils encountered, as well as an illustration of the soil strata breaks, refer to the LOTB attached to this report in Appendix III.

6. GROUNDWATER

Groundwater was not observed in the augured portion of the borings conducted for this study or in the existing channel during the investigation (January 29, 2020.) Measurement of groundwater was not possible below approximately 20 ft depth (elevation 489.3 feet in boring R-20-006 and elevation 492.9 feet in boring R-20-007) due to use of rotary drilling techniques below this depth and time restrictions on leaving the holes open. Based on the observations, the design water elevation is conservatively set at elev. 493.

Groundwater levels can be expected to vary with the level of precipitation, irrigation and other factors. The adjacent golf course and reservoir could be a significant source of groundwater.

For a dry season, construction seepage is expected to be generally minor and limited to nuisance water within the upper 20 ft below existing grade. Groundwater may exist below this depth or as possible isolated perched zones above the weathered rock. Excavations in granular soil below groundwater would be expected to encounter heavy seepage. Seepage within underlying weathered rock would be expected to be minor, but could be locally heavy where fractured.

7. AS-BUILT FOUNDATION

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

8. SCOUR EVALUATION

Alluvium and fill soils are considered susceptible to scour. The underlying weathered rock is considered moderately resistant to scour, but would be susceptible to scour from concentrated flows, especially near the top of the unit. WRECO evaluated the potential scour elevations for the proposed replacement bridge which are summarized in Table 4 (*DRAFT Memorandum;* August 20, 2020).

Table 4. Scour Data Table

Support No.	Long-term (Degradation and Contraction) Scour Elevation (ft)	Short-term (Local) Scour Depth (ft)
Southeast Abutment	505.0	5.6
Northwest Abutment	505.0	5.0

9. CORROSION EVALUATION

The California Department of Transportation (Caltrans) *Corrosion Guidelines*, version 3.0 dated March 2018 has the following definition of corrosive soils:

"For structural elements, the Department considers a site to be corrosive if one or more of the following conditions exists for the representative soil and/or water samples taken at the site:

- *Chloride concentration is 500 ppm* [parts per million] *or greater,*
- *Sulfate Concentration is 1,500 ppm or greater,*
- *pH is 5.5 or less."*

In addition to the conditions listed above, The California Amendments to AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 8th Edition, Section 10.7.5 considers a site corrosive if the additional condition listed below exists for the representative soil and/or water samples taken at the site:

- Minimum resistivity of 1100 ohm-cm or less.

WRECO performed corrosive potential testing for this study on recovered representative soil samples. The results are presented in Table 5 below:

Table 5. Soil Corrosion Data

Boring ID	Depth (ft)	Minimum Resistivity (ohm-cm)	Soil pH	Chloride Content (ppm)	Sulfate Content (ppm)
R-20-001	38	1100	7.67	9.1	13.8
R-20-002	30	540	7.51	149.6	27.1

Based on the corrosive potential testing results, the site soils are considered corrosive to buried metal and concrete as defined by the Caltrans *Corrosion Guidelines*.

10. SEISMIC DESIGN INFORMATION AND RECOMMENDATIONS

10.1. Ground Motion

The Project site is located in a seismic area of California. Potential geologic and seismic hazards for the site include seismic shaking (ground motion), subsidence, and seismically-induced settlement. A seismic study was performed to develop seismic design parameters for the site following the Caltrans' *Seismic Design Criteria* (SDC) Version 2.0 (2019) "Memos to Designer (MTD) Section 20" and design tools outlined in the Caltrans' *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendation* (2012), a seismic analysis was performed for this structure to develop seismic design parameters and to identify potential seismic hazards such as liquefaction or lateral spreading.

A shear wave velocity for the upper 100 feet of the soil/rock profile (V_{S30}) of 442 and 348 meters per second (1449 and 1143 feet per second) was calculated for boring R-20-001 and R-20-002. For design a V_{S30} of 348 m/sec is considered appropriately conservative.

Caltrans SDC 2.0 was adopted September 1, 2019. A major change in SDC 2.0 is the construction of the *Design Spectrum*. Previously, the *Design Spectrum* was constructed using the envelope of probabilistic and deterministic spectra. For SDC 2.0, the *Design Spectrum* is based on the USGS 975-year uniform hazard spectrum only. Effective December 1, 2019, the USGS hazard spectrum is based on the 2014 Nation Hazard Map per memorandum from the State Bridge Engineer. The updated *Design Spectrum* continues the use of near fault adjustment and basin amplification factors.

Based on the information presented above, the peak ground acceleration (PGA) for the site is estimated to be 0.68g (g is the acceleration due to gravity) with a peak spectral acceleration of 1.69g at a period of 0.3 seconds. For wall design a horizontal seismic coefficient (K_h) equal to 1/3 PGA (generally capped at 0.2 by Caltrans) should be used for wall designs.

The seismic design data is listed in Table 6.

Table 6. Seismic Analysis Site Data

Site Location:	Latitude: +37.63781
	Longitude -121.76364
Estimated Site Shear Wave Velocity (V_{S30})	348 meters per second (1143 feet per second)
Peak Ground Acceleration (PGA)	0.68g
Mean Site-source Distance	15.7 km (9.8 miles)
Soil Classification	S1

The ARS curve is presented as Figure 3, located in Appendix I. The seismic analysis is summarized and attached in Appendix V.1.

10.2. Ground Surface Rupture

The Project site is not within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone (AP Zone) and no faults with Holocene or more recent evidence of movement are shown on the California Fault Activity Map (CGS, 2010) or in the USGS Fault and Fold Database. However, projections of active fault traces from mapped Alquist-Priolo Earthquake Fault Zones would pass close to the site. The nearest AP Zone is located approximately 2.2 miles to the northeast of the site and the mapped fault traces appear to align with the course of Dry Creek and the base of hills that run northeast to southwest near the Project site. The active traces belong to the Los Positas Fault to the east of the site and the fault within approximately 2.2 miles of the site is shown on the California Fault Activity Map (CGS, 2010) as having evidence of movement in the Late Quaternary. The trace of the Los Positas Fault at / near the site is shown on geologic mapping as concealed below the alluvium along the course of Dry Creek (Dibblee, 2006) or shown passing the site several hundred feet to the north (USGS, 1996 and USGS 2017).

The exposed sedimentary rock downstream of the bridge is dipping at approximately 20 degrees to the north and is isolated from other rock outcroppings in the general vicinity. This may indicate deformation related to the fault.

Fault displacement was recorded on the Los Positas Fault after the January 1980 earthquakes that struck near Livermore California. Displacements were recorded at Miners Road and Grant Line Road to the northeast of the Project site (USGS, 1980.) Recorded movements were small on the order of 0.5 to 1.5 mm.

While the section of the Los Positas Fault Zone that crosses through or near the Project is not included in an AP Zone and is not listed as active it is considered prudent to provide some provision for small movements through the project site. It is recommended to provide provision to accommodate 0.25 inches of horizontal or vertical motion.

10.3. Liquefaction Evaluation

The Project is located in an identified Liquefaction Seismic Hazard Zone (CGS, 2008) indicating the site likely has soil and groundwater conditions conducive to liquefaction (See Figure 4). Both liquefaction potential and dry dynamic settlement were evaluated for the proposed site’s soils. Liquefaction is the process in which the seismic shear waves cause an increase in the pore water pressure in a cohesionless (sand and some non-cohesive silts) soil strata. This increase in pore water pressure reduces the effective stress that confines the soil. The reduction in effective stress causes a reduction in the shear modulus of the soil, which in turn, results in increased soil deformation.

Also associated with liquefaction is a loss in bearing strength. In the case of full liquefaction, when the increase in pore water pressure reduces the confining stress to zero, the soil experiences a full loss of strength and undergoes large viscous deformations. Lateral spreading (large lateral deformations) are possible when liquefaction occurs in ground having even minimal slope. Primary factors that can trigger liquefaction are moderate to strong ground shaking, relative clean and loose granular soils, and saturated soil conditions. Liquefaction is generally limited to the upper saturated 50 ft of ground surface due to the increasing overburden pressure with depth.

Dry loose to medium dense cohesionless soils when subjected to seismic shear waves compact in place, similar to being compacted with a vibratory roller. This is known as dry dynamic settlement. The energy of the seismic event reorganizes the grains to a more-dense state and subsequently causes a reduction in the overall volume resulting in a settlement of the soils. Dry dynamic settlement is known to occur at any depth in loose sands, as loose sands tend to settle and densify during dynamic shaking.

Dense to very dense sands with and gravely silt was encountered within approximately 5 ft depth in both borings and groundwater was generally absent in the upper 20 ft of the site during the field investigation. Below approximately 30 ft depth in both borings, hard clay (decomposed Claystone) was encountered. Based on these site conditions, the potential for liquefaction and dry dynamic settlement to affect the proposed bridge does not exist.

10.4. Other Seismic Hazards

The areas around the Project site are mapped as within a Landslide Seismic Hazard Zone including the hills to the southeast and east, existing landslides are mapped to the southeast (CGS 2008). The channel bank north and downstream of the bridge has been over steepened and is near vertical in some areas. Alluvial soils overlying weathered rock was exposed in this area. Based on the potential seismic shaking and steepness of the channel slope downstream of the existing bridge, the risk of seismic-induced slope failures is considered high. Other portions of the Project site have more level terrain and the potential for seismically-induced slope failures east and south of the bridge is considered low.

11. FOUNDATION RECOMMENDATIONS

The soils encountered in the two borings performed within the proposed structure limits indicate dense granular soils were encountered at shallow depth and weathered rock was observed outcropping within the limits of the proposed northern abutment. WRECO scour elevations indicate only elev. 505.0 total scour is predicted for the propose bridge. Based on the above, spread footing foundations appear possible founded below predicted scour and cast-in-drilled-hole piles are also feasible. Driven piles are not considered suitable due to hard driving at relatively shallow depth, which would limit penetration.

Even though the site is not mapped within an Alquist-Priolo Special Studies Zone and fault motion recorded on the Los Positas Fault is minimal, it is recommended that the bridge be single span and to provide an allowance for up to 0.25 inches of differential horizontal or vertical movement between the bridge supports. Preliminary foundation design recommendations for the Project were determined using the 2018 AASHTO *LRFD Bridge Design Specification* (BDS) *with California Amendments* as required by the current Caltrans design policy. The recommendations are presented in the following discussions.

11.1. Preliminary Shallow Foundation Recommendations

Preliminary shallow foundation recommendations have been developed to support preliminary design. These recommendations are provided using the following assumptions:

- Support Number: Abutment 1 and 2
- Foundation Material (Soil or Rock): Soil
- Permissible Settlement: 2 in
- Resistance Factor (Strength) - φb: 0.45
- Resistance Factor (Seismic) - φb: 1.00
- Bridge Width: 60 ft
- Bottom Footing Elev. 499.0

A summary of preliminary shallow foundation capacities is provided in Table 7 for various effective footing widths from 5 to 10 ft.

Table 7. Preliminary Foundation Data For Abutments

No.	Effective Footing Width	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B' (ft)	*qn* (ksf)	*qpn* (ksf)	*qR* (ksf)
1	5.0	36.9	8.4	16.6
2	6.0	40.7	6.7	18.3
3	7.0	45.4	6.1	20.0
4	8.0	48.4	5.6	21.8
5	9.0	52.2	5.3	23.5
6	10.0	56.0	4.9	25.2

11.2. Preliminary Deep Foundation Recommendations

The following preliminary CIDH pile recommendations are provided for preliminary design. These recommendations were developed with an assumed cut-off elevation of 503 ft.

Table 8. Preliminary Foundation Design Data Sheet

General Foundation Design Data Sheet							
Support	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	No. Piles per Support
				B	L		
Abut 1	24" CIDH	518.53	503	N/A	60±	2	xx
Abut 2	24" CIDH	517.17	503	N/A	60±	2	xx

Table 9. Preliminary Foundation Design Recommendations

<table>
<tr><th rowspan="3">Support Location</th><th rowspan="3">Pile Type</th><th rowspan="3">Cut-off Elevation (ft)</th><th colspan="2">Service-I Limit State Load per Support (kips)</th><th rowspan="3">Total Permissible Support Settlement (inches)</th><th colspan="4">Required Nominal Resistance (kips)</th><th rowspan="3">Design Tip Elevations (ft)</th><th rowspan="3">Specified Tip Elevation (ft)</th><th rowspan="3">Required Nominal Driving Resistance (kips)</th></tr>
<tr><th rowspan="2">Total</th><th rowspan="2">Permanent</th><th colspan="2">Strength</th><th colspan="2">Extreme Event</th></tr>
<tr><th>Comp (φ = 0.7)</th><th>Tension (φ = 0.7)</th><th>Comp (φ = 1.0)</th><th>Tension (φ = 1.0)</th></tr>
<tr><td>Abut 1</td><td>24” CIDH</td><td>503</td><td>xx</td><td>xx</td><td>2</td><td>400</td><td>N/A</td><td>N/A</td><td>N/A</td><td>457 (a-1)</td><td>xx</td><td>N/A</td></tr>
<tr><td>Abut 2</td><td>24” CIDH</td><td>503</td><td>xx</td><td>xx</td><td>2</td><td>400</td><td>N/A</td><td>N/A</td><td>N/A</td><td>457 (a-1)</td><td>xx</td><td>N/A</td></tr>
</table>

Notes:

1. Design tip elevations are controlled by (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
2. The CIDH Specified Tip Elevation shall not be raised.
3. Design Tip Elevation for Lateral Load is typically provided by the SD.

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The CIDH pile lateral capacity analysis was performed using Ensoft, Inc.'s computer program LPile to predict how the foundation soils at the bridge site deform and deflect in response to lateral load. The soil is modeled using lateral load-transfer and deformation curves (p-y curves) for the proposed Project site.

Table 10. Preliminary LPILE Input Parameters

Support Location	Top Elevation (ft)	Bottom Elevation (ft)	Layer Thickness (ft)	Soil Type	γ (pcf)	φ (°)	c (psf)	k [1] (pci)	ε_{50}
Abutments	510	482	28	Sand (Reese)	65	40	xx	125	xx
	482	441	41	Clay	70	xx	4000	2000	0.004

11.3. Abutment Backfill

The abutments and retaining walls should be backfilled according to the construction techniques described in Section 19-3.03E "Structure Backfill" of the Caltrans 2018 or most recent *Standard Specifications*. Backfill should be compacted to a minimum of 95 percent of the maximum wet density as determined by California Test Method (CTM) 216 or ASTM 1557.

In addition to the minimum requirements stated in Section 19 "Structure Backfill" of the Caltrans 2018 or most recent *Standard Specifications*, the following minimum material requirements should also be met in order to use the provided soils design information for structure backfill material:

- Maximum Plasticity Index (PI) of 6
- Minimum Sand Equivalent of 20
- Minimum R-value of 30

As long as the abutments and retaining walls are backfilled using material which conforms to the above requirements, the following soil properties and equivalent fluid pressures may be used to determine the soil loading on the abutments and retaining walls:

- Total Unit Weight of Soil (γ_{tot}) = 120 pounds per cubic foot (pcf)
- Internal Angle of Friction (ϕ) = 34°
- Undrained Shear Strength (c) = 0 pounds per square foot (psf)
- Active Equivalent Fluid Pressure = 34 psf
- At-rest Equivalent Fluid Pressure = 53 psf
- Passive Equivalent Fluid Pressure = 350 psf

These abutments and retaining walls are considered yielding structures and will allow the development of active earth pressures. They should be designed using active equivalent fluid pressures to determine the lateral soil loading.

Drainage behind all abutment retaining walls is essential for the stability of the structures. Many retaining structure failures are due to a buildup of hydrostatic water pressure behind the wall; loading which the structure was not designed to accept. As a rule of thumb, 1 ft of water is equivalent to 2 ft of soil loading, so if only two-thirds (2/3) of the height of the wall is subjected to hydrostatic pressure, the structure is subjected to an additional equivalent soil loading 1.33 times the design height.

Every retaining wall should have a drainage system similar to the one on the Caltrans 2019 *Standard Plan,* B0-3, Bridge Detail 3-1, or an approved geocomposite chimney-type drain material. This drain should provide positive drainage to daylight and be maintained to prevent a debris clog and buildup of hydrostatic pressure.

11.4. Approach Fill Earthwork

Prior to grading for the planned fill, it is recommended that any trash, debris, and vegetation be removed. Depressions left by any such removal should be backfilled in accordance with the Caltrans *Standard Specifications* or recommendations made in this report. Loose or soft soil identified during grading operations should be removed from within the embankment footprint, and a firm subgrade should be exposed prior to placing new fill material. It is not expected all materials generated from new cuts and/or excavations will meet the requirements for structure backfill for the Project and therefore, imported fill material will likely be required. All fill shall have a minimum Resistance R-value of 30.

In areas where new fill is to be placed onto existing fill slopes or natural slopes exceeding 5H:1V (Horizonal: Vertical), a full bond between the two materials will need to be developed by placing the new fill on discrete horizontal benches that are cut fully into the existing slope and below any loose/soft or otherwise unsuitable materials (as per Section 19 of Caltrans *Standard Specifications*). In areas where existing utilities are present within the limits of the embankment fill, complete removal of the utilities is recommended, followed by subsequent replacement with backfill compacted to 95% of the maximum density (based on ASTM 1557). Placement of embankment fill can then proceed when these areas are brought up to grade.

If the native soils from the excavation appear to be saturated from either perched or transient groundwater and/or have a water content above the optimum moisture content as determined by ASTM D1557 methods, and if these excavated soils are to be used for backfill of the excavated trench, they must be brought to the optimum moisture content to obtain the minimum required compaction. Typical methods of achieving this are drying soils in wind rows or blending them with dry import fill to reduce the overall moisture content of the blended fill material.

11.4.1. Expansive Material

Expansive soil (EI $\geq$ 50 and SE $\leq$ 20) should not be used as fill within 5 ft of the back of the abutment wall/wingwall or in any portion of the abutment front slope.

11.4.2. Approach Fill Settlements

The proposed approaches are generally near existing grade with low fill embankments on the order of 5 ft or less planned near the ends of the bridge. Settlement of these fills is expected to be substantially complete by the end of construction and no appreciable long-term settlement is expected for these low embankments founded on weathered rock.

11.5. Approach Cuts

The widening of the approach roadway north of the bridge will require cutting into the existing slope west of the existing roadway. Based upon visual observations and the existing slopes made during field visits, we anticipate the slope can be cut at a 1.5H:1V to facilitate the roadway widening and remain reasonably stable. It should be noted near surface weathered rock was observed just downstream of the bridge and may be encountered in planned cuts. The exposed rock dipped to the north and may present an adverse bedded condition (dipping out of slope) along the planned cuts. Overlying materials are gravels and sands, which may ravel or be easily eroded. We also anticipate there will be areas of more weathered/less competent materials that will require shallower slopes. As these determinations can only be made in the field during grading, we recommend that a representative of WRECO be present during grading to document to verify the soil and rock conditions present in the slope cut and provide additional recommendations as required.

11.6. Approach Pavement Sections

New structural pavement sections will be constructed for the bridge and its approaches. The following table, 11, provides the design Traffic Indices' (TI), design R-value, and structural pavement Hot Mix Asphalt (HMA) and Class 2 Aggregate Base (AB) thicknesses.

Table 11. New HMA-AB Flexible Structural Pavement Sections

Design TI	Design R-value	HMA Thickness (ft)	Class 2 AB Thickness (ft)
5.5	30	0.25	0.55
6.0	30	0.25	0.70
6.5	30	0.30	0.70
7.0	30	0.30	0.85
7.5	30	0.35	0.85
8	30	0.40	0.90
Notes: TI=Traffic Index; HMA=Hot Mix Asphalt; AB=Aggregate Base			

Pavement design and construction should conform to the requirements of the Caltrans Standard Specifications, 2018 edition. All native material or import fill used below the new pavement sections should possess an R-value equivalent to or greater than the design R-value (30). All trench backfill for utilities and pipes underlying paved areas should be properly placed and compacted to at least 95 percent compaction (ASTM D1557 or ASTM D1557 or CTM 216) to provide a stable pavement subgrade. The upper 30 inches of all pavement subgrades should be moisture conditioned and compacted to at least 95 percent relative compaction (ASTM D1557), per Caltrans Standard Specifications. Existing pavement section materials (AC, PCC, and AB) can be recycled as aggregate base for the new pavement sections. Copies of the structural pavement calculations are included in Appendix V.3.

12. ADDITIONAL CONSIDERATIONS

All excavation and backfill work shall be performed in accordance with Section 19, Earthwork, of the Caltrans *Standard Specifications*. Groundwater can be expected and excavation below creek water level will likely require dewatering, cofferdam construction, seal courses, or other means to control groundwater. Soils within the depth of anticipated excavations are generally rippable, but would likely require shoring to prevent collapse. Encountered materials appear drillable by typical large foundation drilling equipment, tough the weathered rock becomes more competent with depth and difficult drilling conditions may be encountered requiring coring buckets, downhole hammers, or other means to excavate. It should be noted Portland cement concrete pavement was encountered below the existing asphalt concrete pavement surface and difficult excavation of the road section should be expected.

The site near surface soils identified in the borings and observed within the channel area and banks near the Project are most similar to Type C soils as defined by the California Division of Occupational Safety and Health (CalOSHA). Dense/stiff clay sands, clays, and decomposed rock meet the requirements for Type B soils found starting at 5-10 ft depth in the borings and appear to exist along the slopes adjacent to the road northwest of the existing bridge and along the north bank of the creek downstream of the bridge. It can be expected temporary excavations in surficial soils and above decomposed rock in the channel can be made with 1.5H:1V (H:V) or flatter side slopes above groundwater. Temporary cuts within underlying Type B soils and decomposed rock can be expected to be made with 1:1 of flatter side slopes. Cuts in decomposed rock can potentially be cut steeper than 1:1, but will required review by a qualified engineering geologist or geotechnical engineer prior to completion of cut and consideration of overlying Type C soils in some locations.

The proposed replacement bridge is close to existing overhead and underground utilities. Existing utilities should be move, de-energized, or protected in-place as appropriate.

Removal of the existing bridge and utilities may disrupt the soil and rock at potential footing elevations. Any soft/disturbed soils should be removed full depth and replaced with granular fill compacted to a minimum of 95% relative compaction per ASTM 1557.

13. LIMITATIONS

This *Preliminary Foundation Report* was prepared in accordance with generally accepted geotechnical engineering principles and practices. No other warranty, expressed or implied, is made as to the conclusions and professional recommendations made in this report.

This *Preliminary Foundation Report* is intended for use with the Arroyo Road Bridge Replacement Project located in Alameda County, California, and any changes in the design or location of the proposed new improvements, however slight, should be brought to our attention so that we may determine how they may affect our conclusions and recommendations. The conclusions and recommendations contained in this report are based upon the data relating only to this specific location and locations discussed herein.

14. REPORT COPY LIST

This *Preliminary Foundation Report* was prepared for Wood Rodgers for use in planning and design of the proposed Arroyo Road Bridge Replacement Project.

15. REFERENCES

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Appendix I. Site Maps

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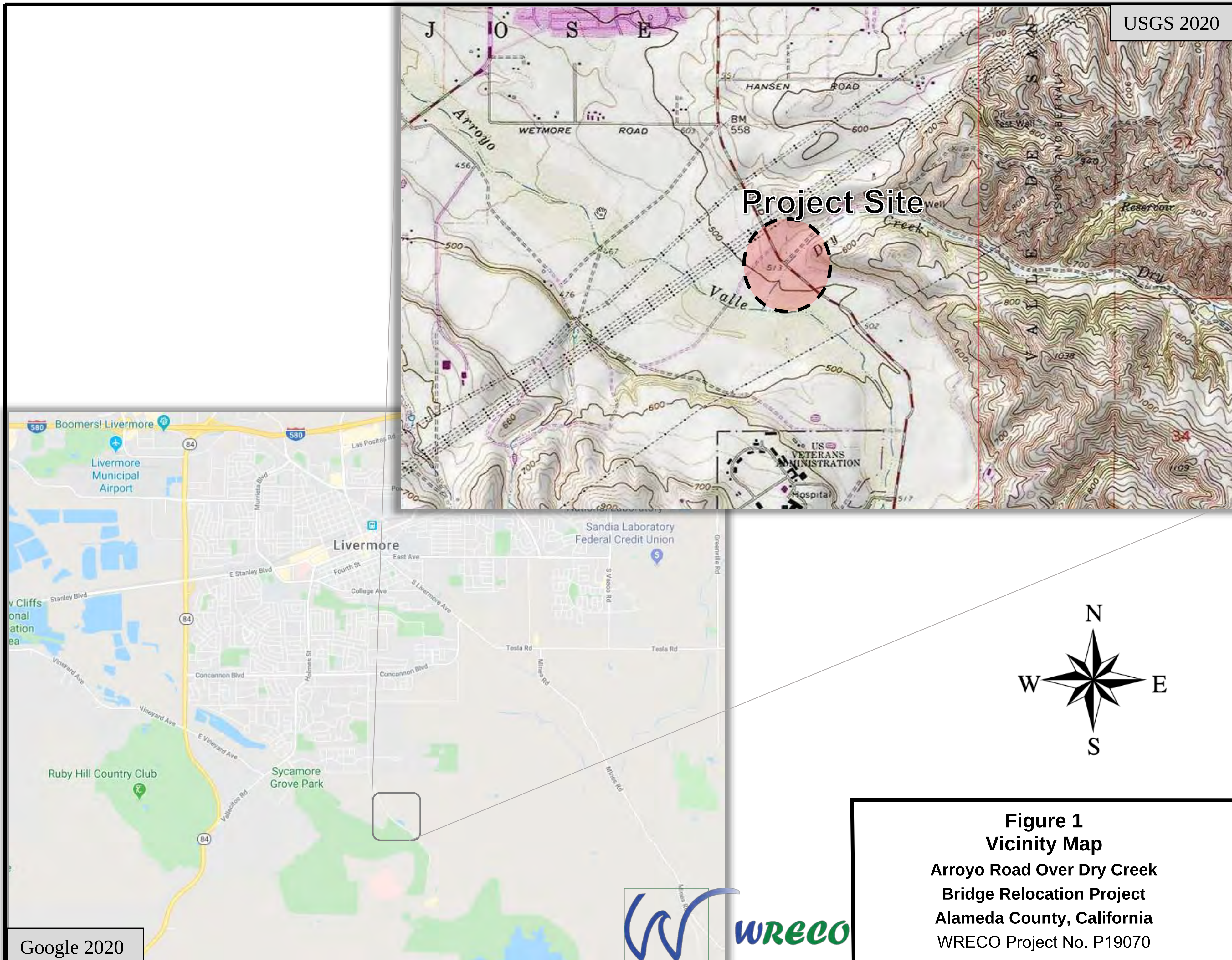
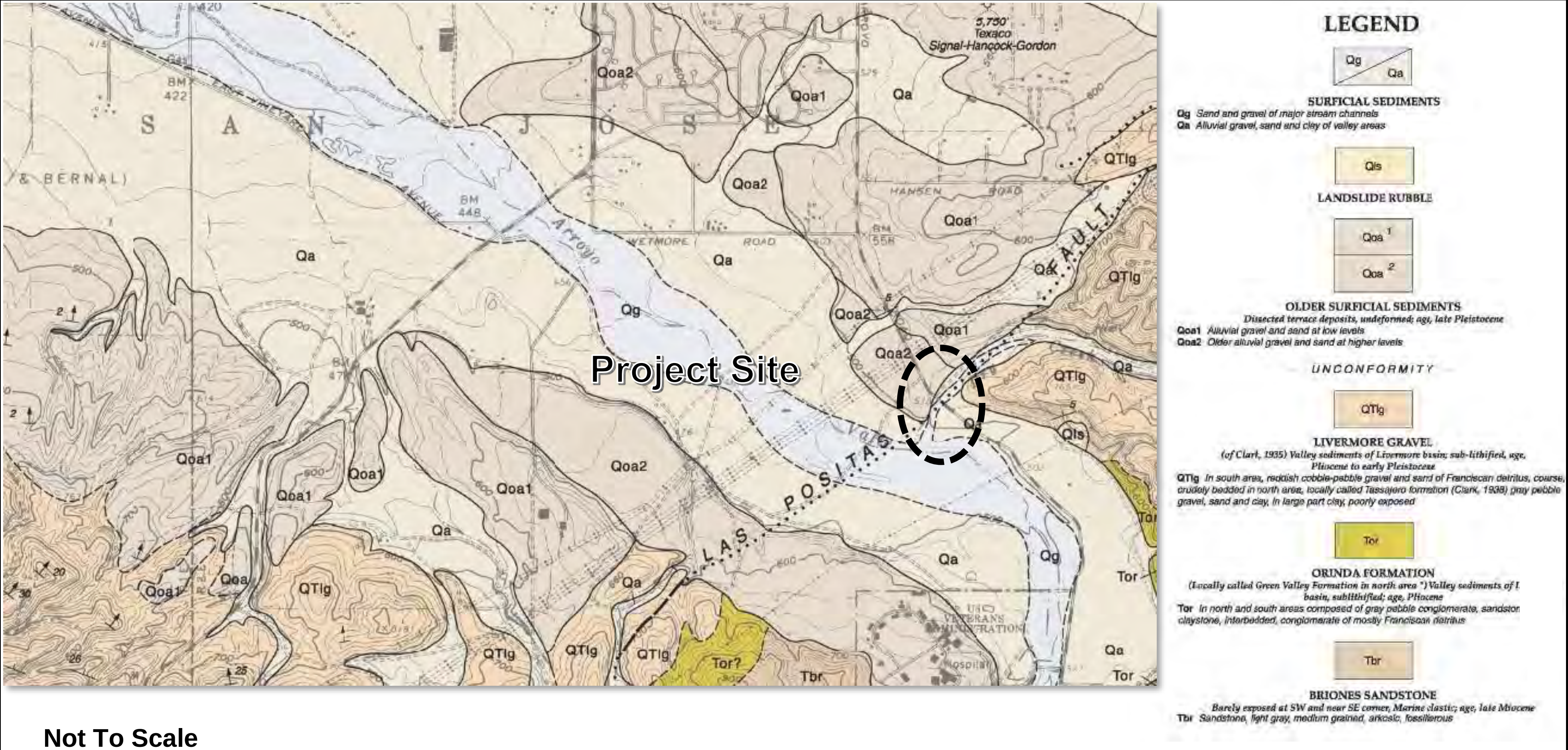


Figure 1
Vicinity Map
Arroyo Road Over Dry Creek
Bridge Relocation Project
Alameda County, California
WRECO Project No. P19070



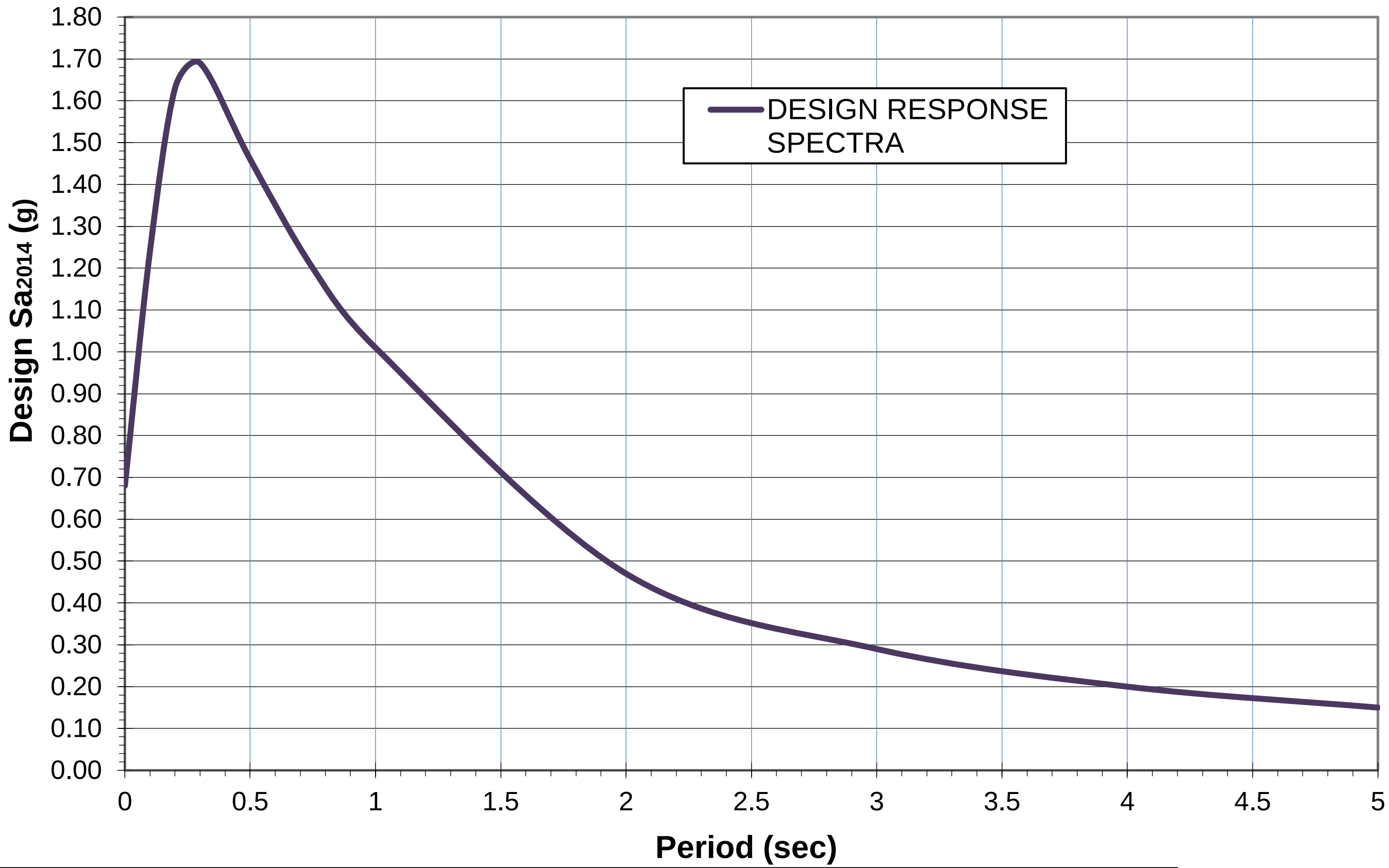
Not To Scale

Reference:
Dibblee, T.W., and Minch, J.A., 2006, Geologic map of the Livermore quadrangle, Contra Costa and Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-196.

N
W E
S

WRECO

Figure 2
Regional Geologic Map
Arroyo Road Over Dry Creek
Bridge Replacement Project
Alameda County, California
WRECO Project No. P19070



Latitude:	37.63781
Longitude:	-121.76364
Vs (m/s):	348

DESIGN RESPONSE SPECTRA

Period (seconds)	Design Sa_{2014} (g)
0	0.68
0.10	1.24
0.20	1.63
0.30	1.69
0.50	1.46
0.75	1.20
1.00	1.01
2.00	0.47
3.00	0.29
4.00	0.20
5.00	0.15

For Design Engineer:

SEISMIC DESIGN: Caltrans Seismic Design Criteria (SDC), Version 2.0 dated April 2019

Deaggregation (based on 2014 hazard)

(a) mean magnitude (for PGA) = 6.71

(b) mean site-source distance (for Sa at 1s) = 15.7 km

Reference: Caltrans ARS Online, v3.0

WRECO

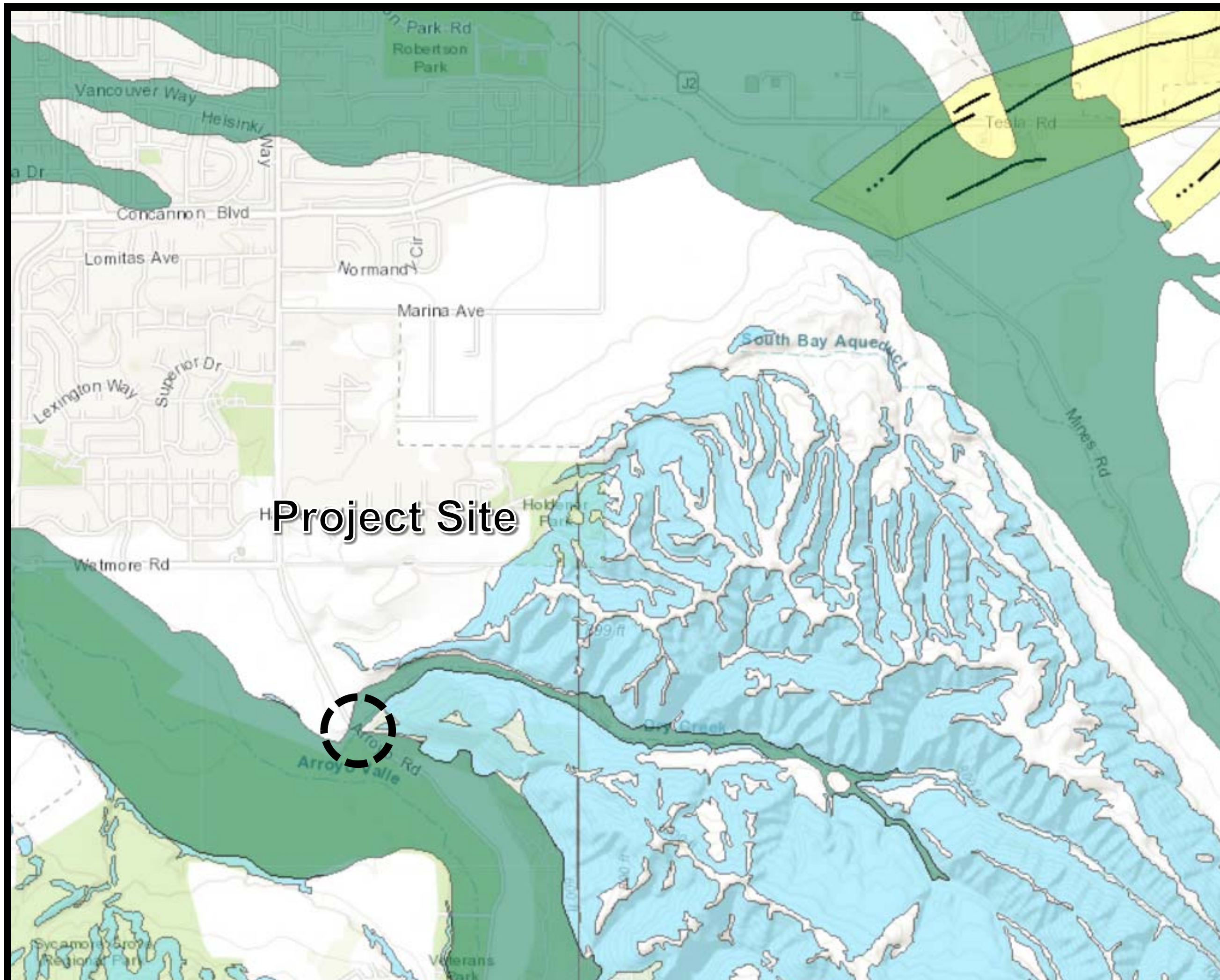
FIGURE 3

Design Response Spectrum (ARS Curve)

Arroyo Road Dry Creek Bridge Replacement

Alameda County

WRECO Proejct No. P19070



LEGEND

Fault Traces

- Accurately Located
- Approximately Located
- Approximately Located, Queried

Fault Zone

Liquefaction Zone

Landslide Zone

Liquefaction Landslide Overlap Zone

Area Not Evaluated for Liquefaction or Landslides

N
W E
S

Not to Scale

Reference:
Earthquake Zones of Required Investigation, California Geological Survey, California Department of Conservation

WRECO

Figure 4
Regional Seismic Map
Arroyo Road over Dry Creek
Bridge Replacement Project
Alameda County, California
WRECO Project No. P19070

Appendix II. Log of Test Borings

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	ALA	XX			

05/13/2021
CERTIFIED ENGINEERING GEOLOGIST DATE

REGISTERED GEOLOGIST DAVID A. KITZMANN No. 2412 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

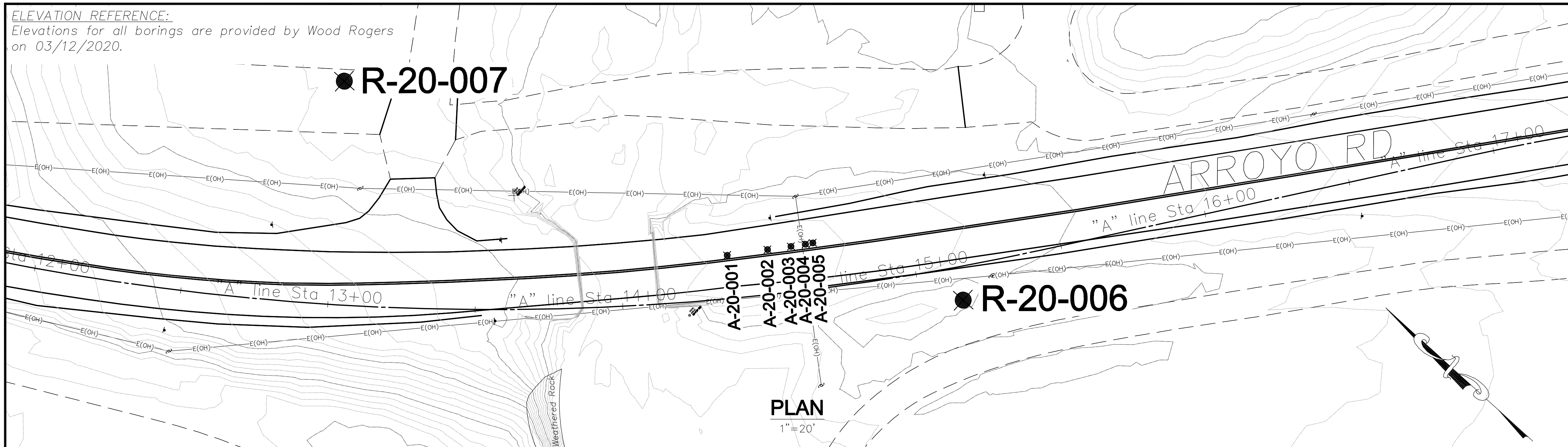
PLANS APPROVAL DATE

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

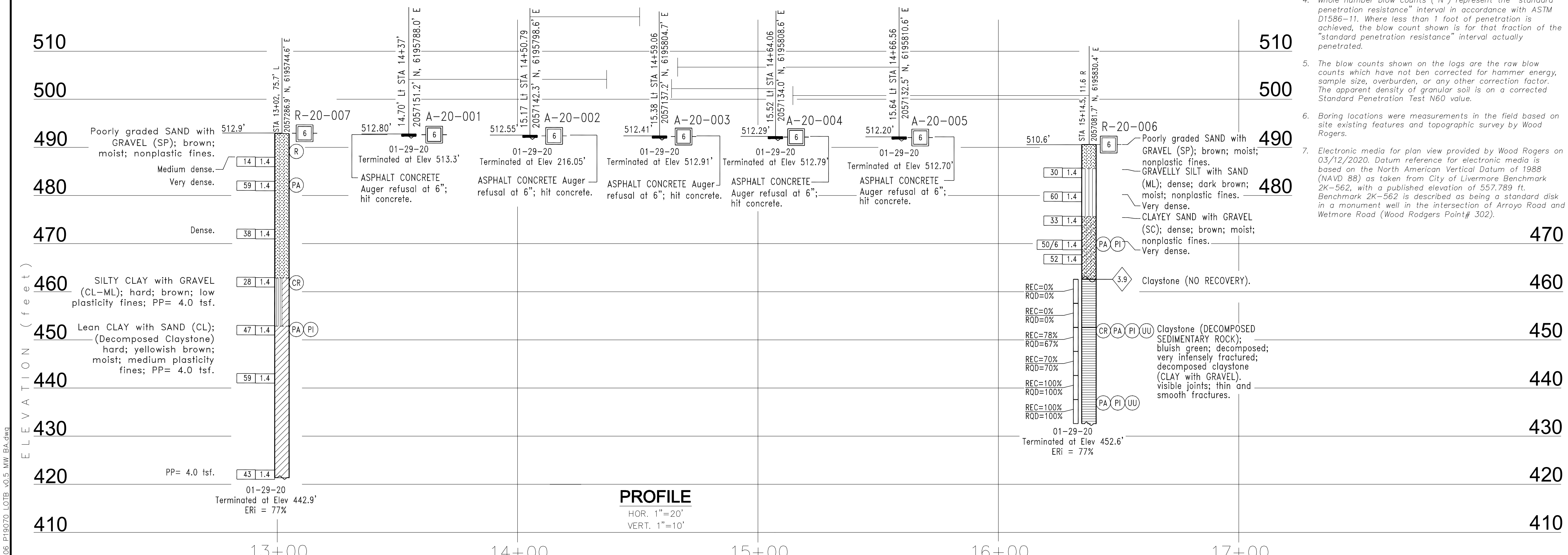
WRECO
7807 LAGUNA BLVD., SUITE 400
ELK GROVE, CA 95758
WRECO JOB NO: P19070

ALAMEDA COUNTY PUBLIC WORKS
399 ELMHURST ST.,
HAYWARD, CA 94544

ELEVATION REFERENCE:
Elevations for all borings are provided by Wood Rogers on 03/12/2020.



PLAN
1"=20'



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

Notes:

1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010 Edition).
2. 1.4" samples were taken using a 1.375" split barrel sampler per Standard Penetration Test (SPT) performed in accordance with ASTM D 1557-11.
3. 1.4" samples were taken with an automated hammer system consisting of a hammer automated of a hammer weight 140 lbs. free falling a distance of 30". Autohammer energy ratio (ETR) measurements indicate an ETR=77%.
4. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with ASTM D1586-11. Where less than 1 foot of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
5. The blow counts shown on the logs are the raw blow counts which have not ben corrected for hammer energy, sample size, overburden, or any other correction factor. The apparent density of granular soil is on a corrected Standard Penetration Test N60 value.
6. Boring locations were measurements in the field based on site existing features and topographic survey by Wood Rogers.
7. Electronic media for plan view provided by Wood Rogers on 03/12/2020. Datum reference for electronic media is based on the North American Vertical Datum of 1988 (NAVD 88) as taken from City of Livermore Benchmark 2K-562, with a published elevation of 557.789 ft. Benchmark 2K-562 is described as being a standard disk in a monument well in the intersection of Arroyo Road and Wetmore Road (Wood Rodgers Point# 302).

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	
FUNCTIONAL SUPERVISOR: DAVID A. KITZMANN	DRAWN BY: OA / MW	FIELD INVESTIGATION BY: O. ADAH	DATE: 1-29-2020
	CHECKED BY: FPT		

PREPARED FOR
ALAMEDA COUNTY
PUBLIC WORKS DEPARTMENT

DAVID A. KITZMANN
PROJECT ENGINEER

BRIDGE NO. XXX-XXXX
POST MILE XX.XX

ARROYO ROAD BRIDGE REPLACEMENT PROJECT
LOG OF TEST BORINGS

OOS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU XXXXXX
EA XXXXXX
FILE => $REQUEST

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 01 OF 01

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

6/12/2020 2021 0506 P19070 LOTB v0.5 MW BA.dwg

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Appendix III. Laboratory Test Results

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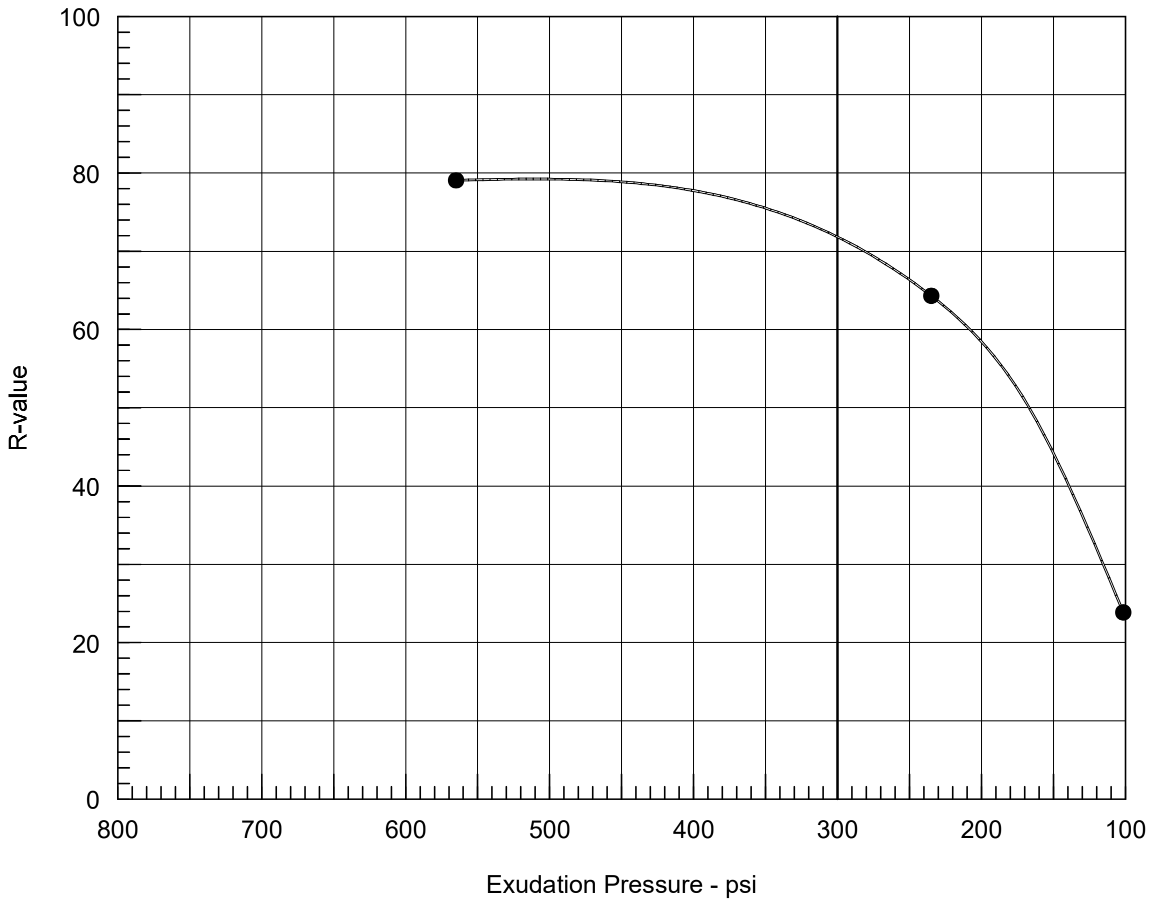
R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	135.8	6.5	48	20	2.43	565	80	79
2	93	134.6	7.6	0	41	2.45	235	64	64
3	12	132.7	8.6	0	105	2.48	101	24	24

Test Results	Material Description
R-value at 300 psi exudation pressure = 72	SILTY GRAVEL with SAND, dark brown

Project No.: 3390.X

Project: WRECO Lab Testing

Source of Sample: P19070 Arroyo Road Bridge **Depth:** 0-5'

Sample Number: R-20-007, S1

Date: 4/2/2020

Tested by: BRL

Checked by: RBL

Remarks:
34% retained on the #4 sieve, sample batched

R-VALUE TEST REPORT

Blackburn Consulting

Figure ______

Unconfined Compression
ASTM D 2166

Project Name:	WRECO P19070 Arroyo Road Bridge
Project Number:	3390.X
Sample ID:	R-20-006, S-9
Type of Sample:	2.4" Core — Depth: 38’
Sample Description:	Weak Rock
Sample Collection Date:	
Day Break:	N/A

Sample Data

Sample Length:	4.91	in	Sample + Tube:	667	g
Diameter:	2.25	in	Tube:	0.00	g
Height-to-Diameter Ratio:	2.19		Sample Weight:	667	g
Sample Area:	3.96	in^2	Wet Density:	130.7	pcf
Sample Volume:	19.4	in^3	Moisture:	5	%
Specific Gravity:	2.65	(assumed)	Dry Density:	124.3	pcf
			Saturation:	41.2	%

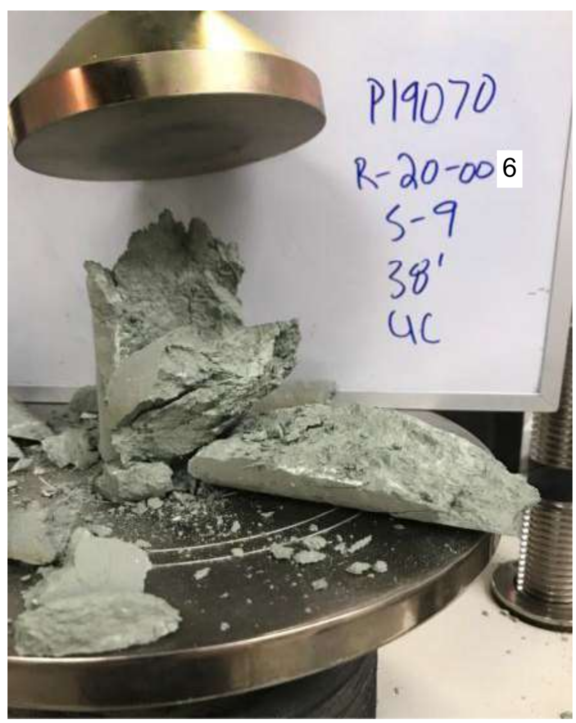
**Moisture content taken after test*

Test Results

Rate of Strain:	0.0491	in/min
Deflection at Max. Load:	0.059	in
Maximum Load:	2,195	lbs
Strain at Failure:	1.19	%
Average cross-sectional area at failure:	4.01	in^2
Compressive Strength:	**39.45**	**tsf**
	547.9	**psi**

Strain Information

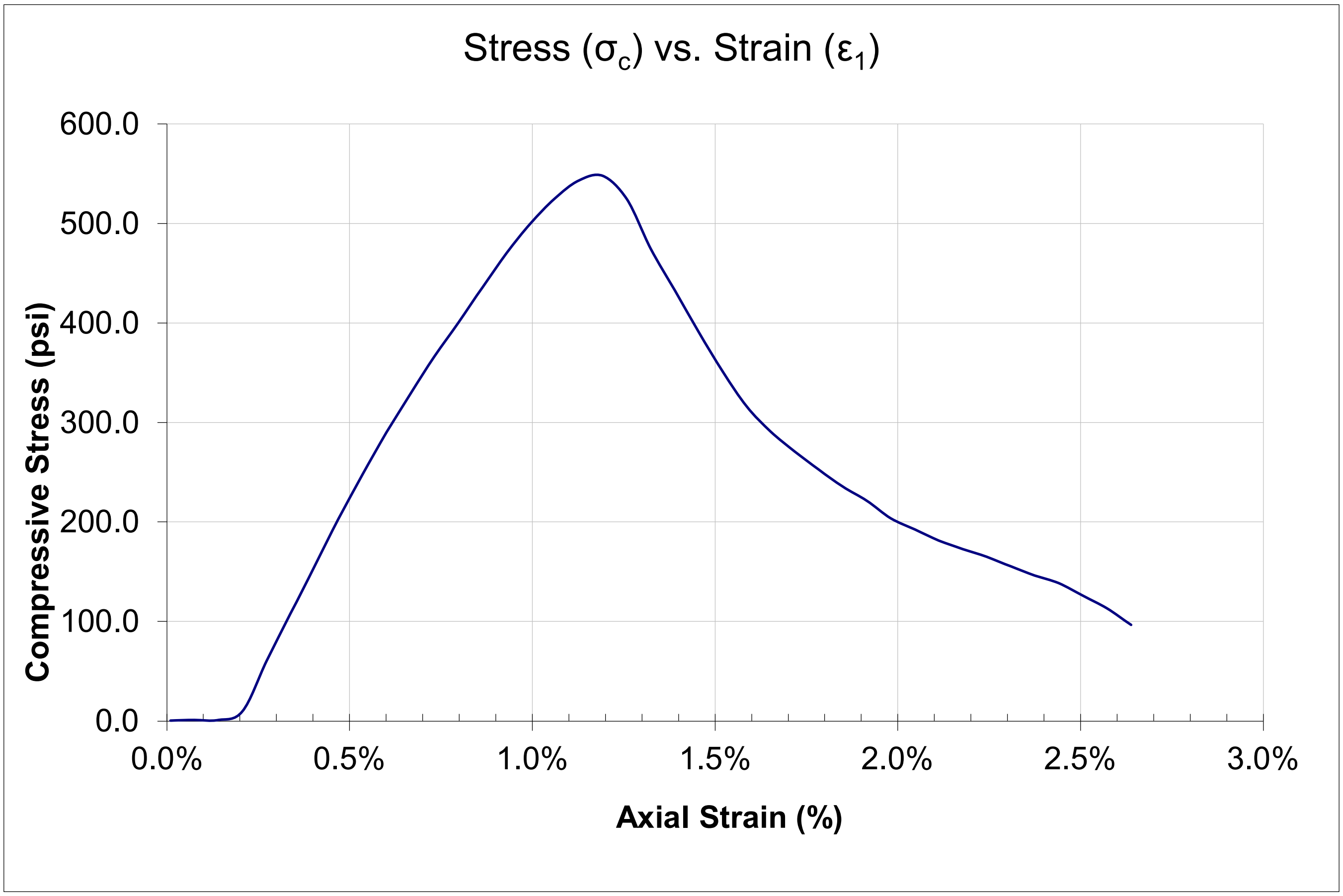
Rate of Strain ½%:	0.025	in/min
Rate of Strain 2%:	0.098	in/min
Strain Rate:	0.049	in/min
15% Strain:	0.737	in



Unconfined Compression
ASTM D 2166

Project Name: WRECO P19070 Arroyo Road Bridge
Project Number: 3390.X
Sample ID: R-20-006, S-9
Type of Sample: 2.4" Core   Depth: 38’
Sample Description: Weak Rock
Sample Collection Date:
Day Break: N/A

Compressive Strength:	**39.45**	**tsf**
	547.9	**psi**



Unconfined Compression
ASTM D 2166

Project Name: WRECO P19070 Arroyo Road Bridge
Project Number: 3390.X
Sample ID: R-20-006, S-12
Type of Sample: 2.4" Core Depth: 53
Sample Description: Weak Rock
Sample Collection Date:
Day Break: N/A

Sample Data

Sample Length:	4.52	in	Sample + Tube:	635	g
Diameter:	2.30	in	Tube:	0.00	g
Height-to-Diameter Ratio:	1.96		Sample Weight:	635	g
Sample Area:	4.16	in^2	Wet Density:	128.6	pcf
Sample Volume:	18.8	in^3	Moisture:	7	%
Specific Gravity:	2.65	(assumed)	Dry Density:	119.9	pcf
			Saturation:	50.9	%

**Moisture content taken after test*

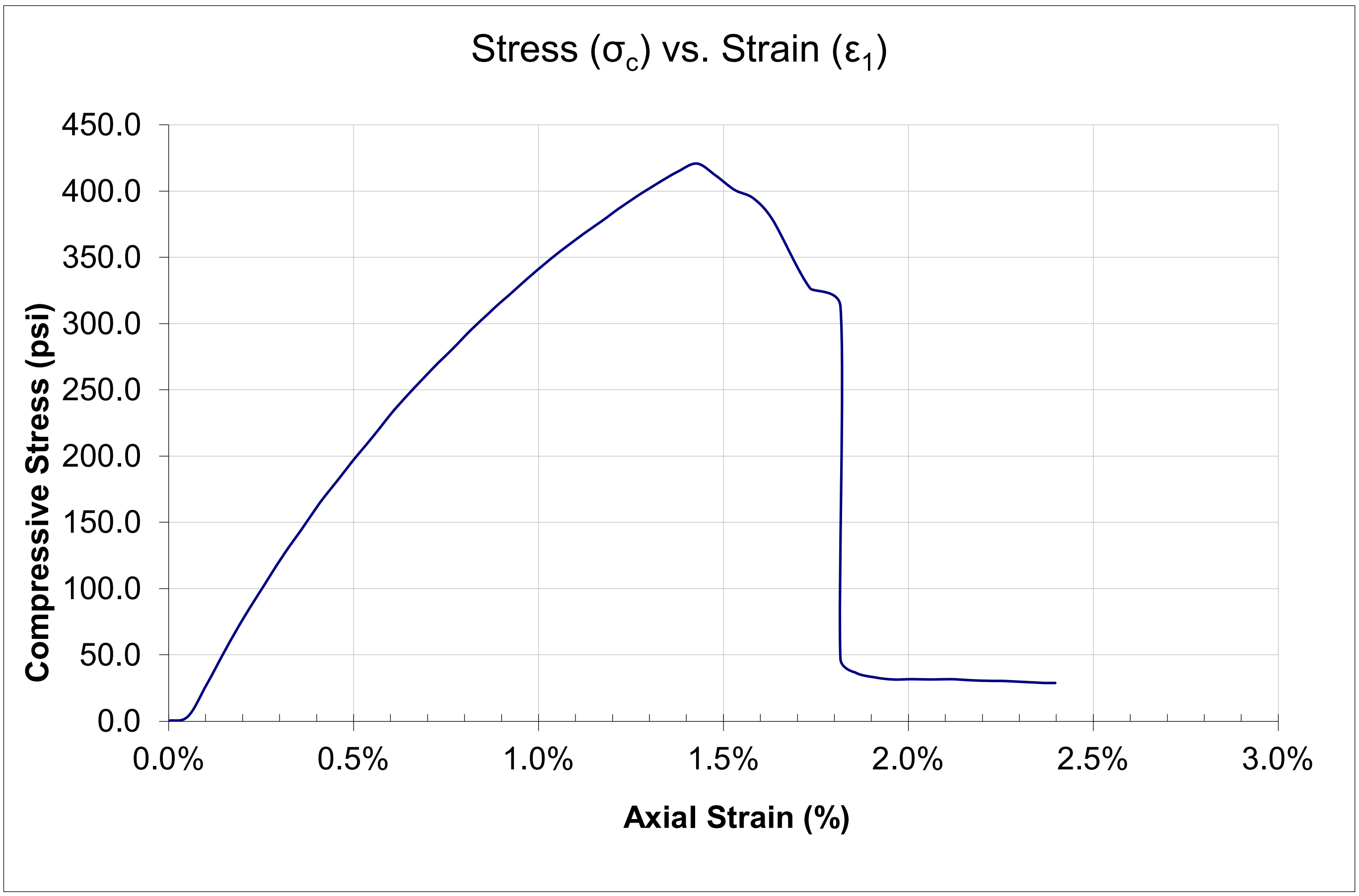
Test Results			**Strain Information**		
Rate of Strain:	0.0452	in/min	Rate of Strain ½%:	0.023	in/min
Deflection at Max. Load:	0.065	in	Rate of Strain 2%:	0.090	in/min
Maximum Load:	1,777	lbs	Strain Rate:	0.045	in/min
Strain at Failure:	1.43	%	15% Strain:	0.678	in
Average cross-sectional area at failure:	4.22	in^2			

Compressive Strength:	**30.30**	**tsf**
	420.9	**psi**

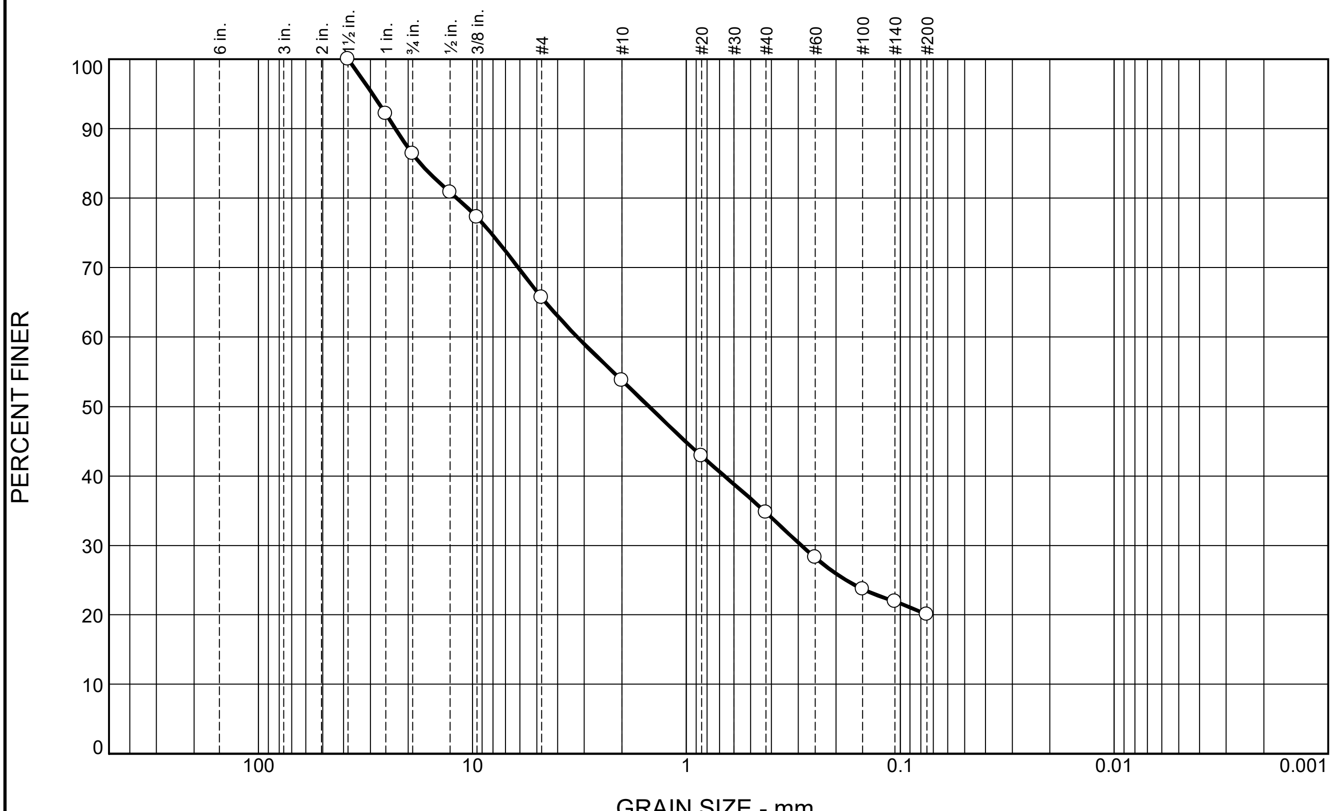
Unconfined Compression
ASTM D 2166

Project Name: WRECO P19070 Arroyo Road Bridge
Project Number: 3390.X
Sample ID: R-20-006, S-12
Type of Sample: 2.4" Core   Depth: 53
Sample Description: Weak Rock
Sample Collection Date:
Day Break: N/A

Compressive Strength: **30.30 tsf**
420.9 psi



Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	14	20	12	19	15	20	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100		
1"	92		
3/4"	86		
1/2"	81		
3/8"	77		
#4	66		
#10	54		
#20	43		
#40	35		
#60	28		
#100	24		
#140	22		
#200	20		

* (no specification provided)

Material Description

CLAYEY SAND with GRAVEL, olive brown

Atterberg Limits

PL= 19 LL= 35 PI= 16

Coefficients

D_{90}= 22.9119 D_{85}= 17.5333 D_{60}= 3.2341
D_{50}= 1.4961 D_{30}= 0.2902 D_{15}=
D_{10}= C_u= C_c=

Classification

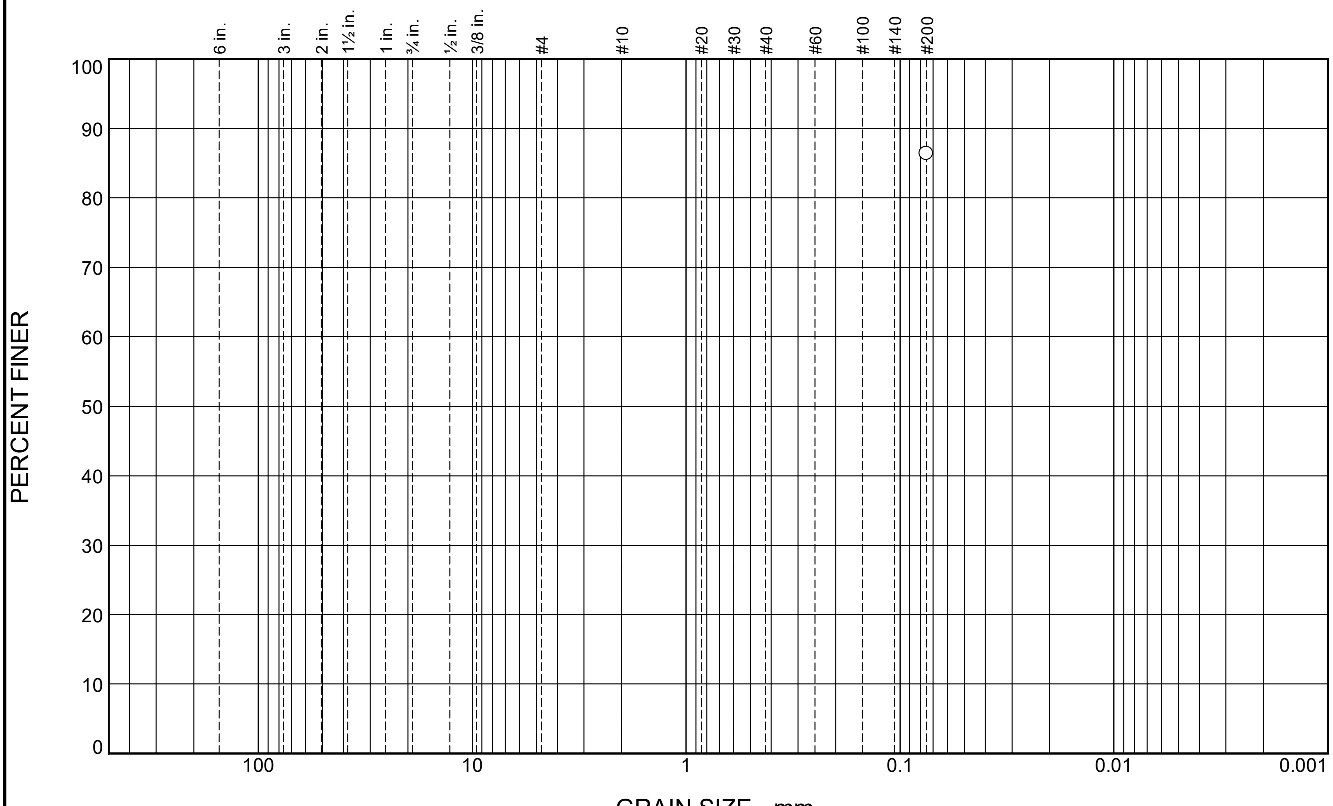
USCS= SC AASHTO= A-2-6(0)

Remarks

Source of Sample: P19070 Arroyo Road Bridge **Depth:** 20'
Sample Number: R-20-006, S5 **Date:**

Blackburn Consulting	**Client:** WRECO
	Project: WRECO Lab Testing
W. Sacramento, CA	**Project No:** 3390.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						86	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	86		

* (no specification provided)

Material Description

Lean CLAY, greenish gray

Atterberg Limits

PL= 16 LL= 43 PI= 27

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

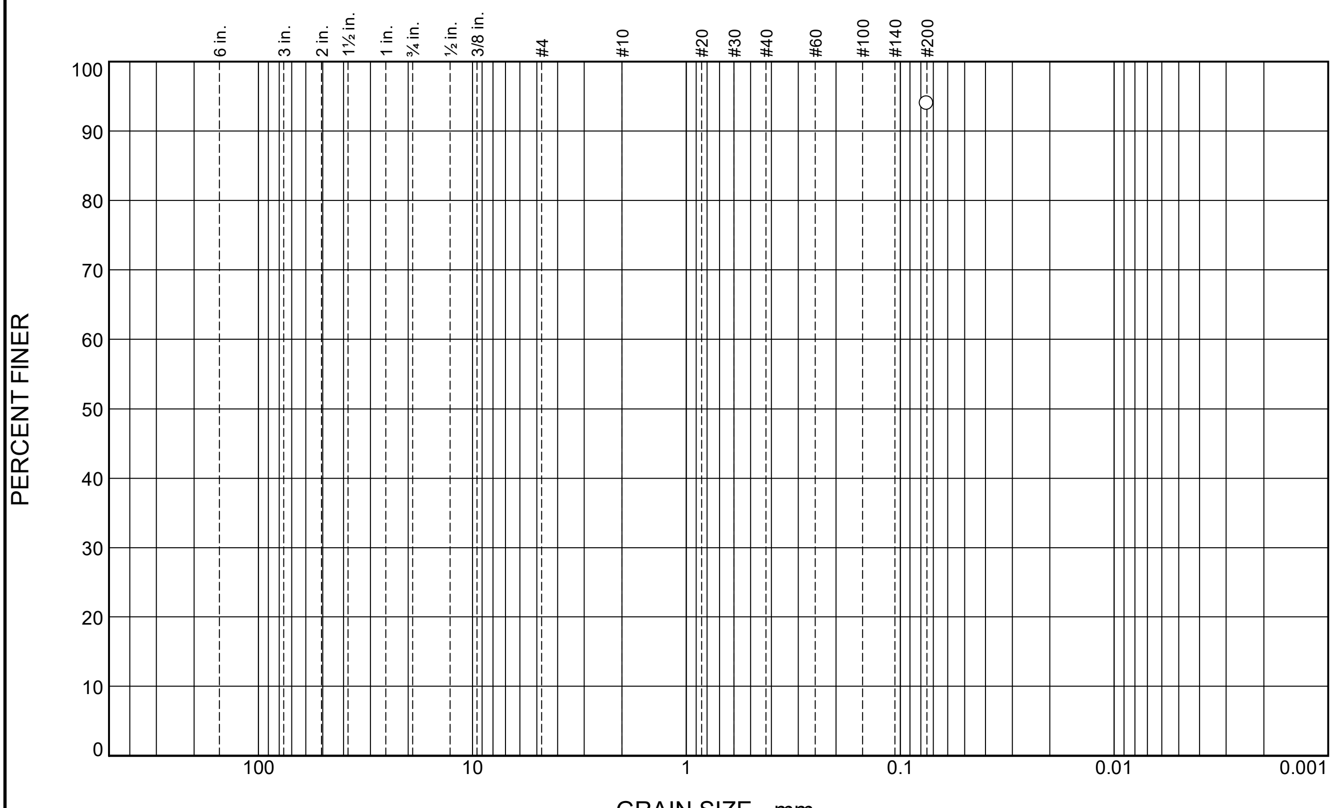
Source of Sample: P19070 Arroyo Road Bridge **Depth:** 38'
Sample Number: R-20-006, S9 **Date:**

Blackburn Consulting

W. Sacramento, CA

Client: WRECO
Project: WRECO Lab Testing
Project No: 3390.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						94	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	94		

* (no specification provided)

Material Description

Lean CLAY, grayish green

Atterberg Limits

PL= 18 LL= 43 PI= 25

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

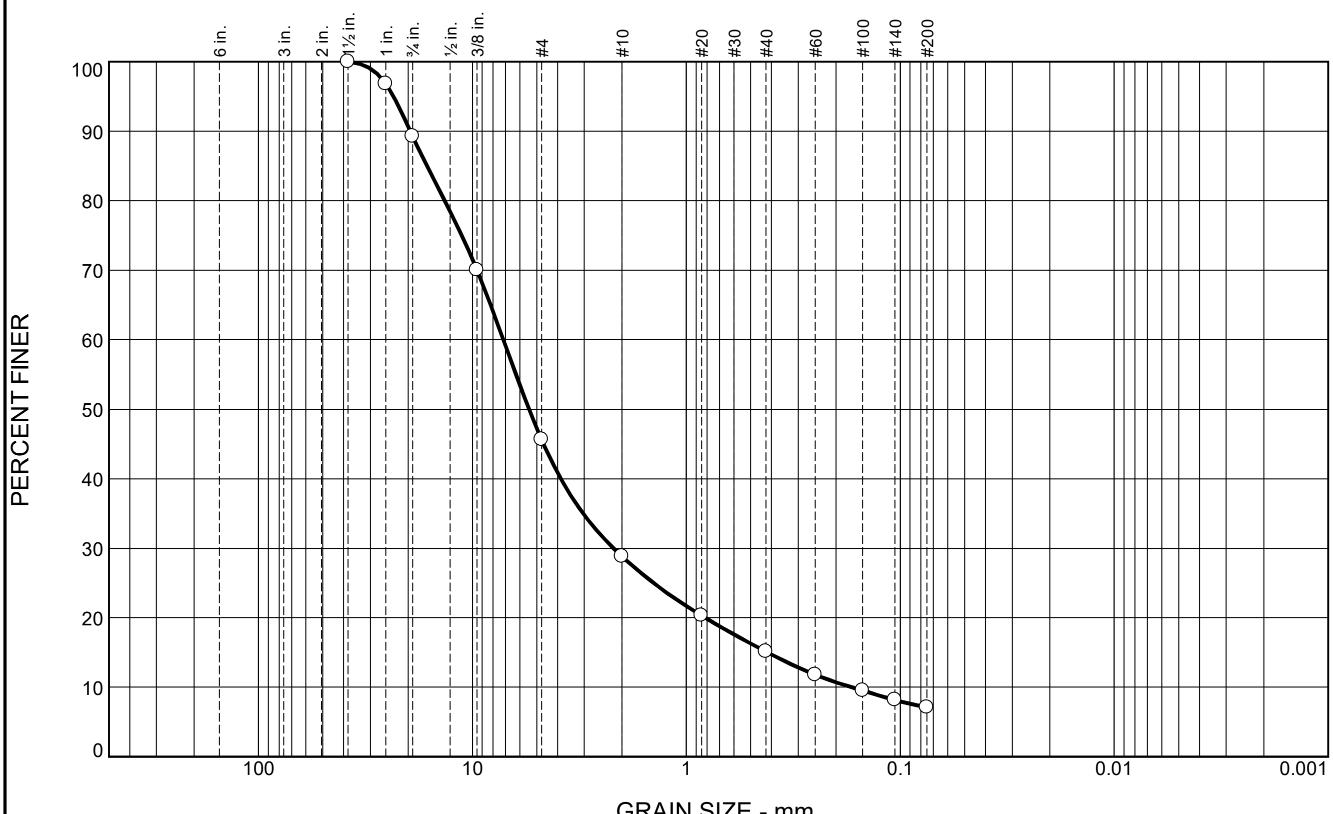
USCS= CL AASHTO=

Remarks

Source of Sample: P19070 Arroyo Road Bridge **Depth:** 53'
Sample Number: R-20-006, S12 **Date:**

Blackburn Consulting **W. Sacramento, CA**	**Client:** WRECO **Project:** WRECO Lab Testing **Project No:** 3390.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	11	43	17	14	8	7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100		
1"	97		
3/4"	89		
3/8"	70		
#4	46		
#10	29		
#20	20		
#40	15		
#60	12		
#100	10		
#140	8		
#200	7.1		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 19.5305 D_{85}= 16.2956 D_{60}= 7.1642
D_{50}= 5.4331 D_{30}= 2.1922 D_{15}= 0.4164
D_{10}= 0.1683 C_u= 42.56 C_c= 3.99

Classification

USCS= AASHTO=

Remarks

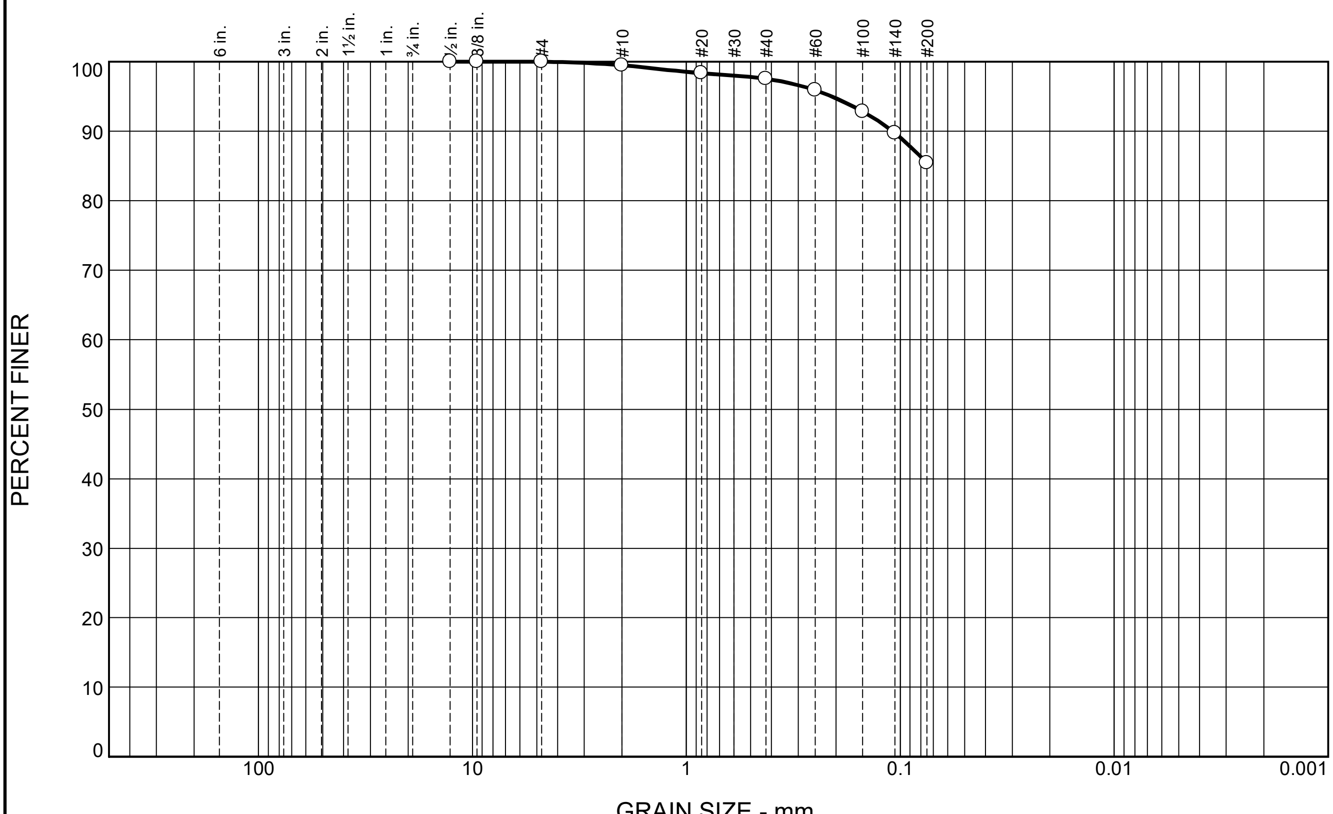
Source of Sample: P19070 Arroyo Road Bridge **Depth:** 10'
Sample Number: R-20-007, S3 **Date:**

Blackburn Consulting

W. Sacramento, CA

Client: WRECO
Project: WRECO Lab Testing
Project No: 3390.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	1	13	85	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	100		
#4	100		
#10	99		
#20	98		
#40	98		
#60	96		
#100	93		
#140	90		
#200	85		

* (no specification provided)

Material Description

Lean CLAY with SAND, yellowish brown

Atterberg Limits

PL= 17 LL= 49 PI= 32

Coefficients

D_{90}= 0.1087 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(28)

Remarks

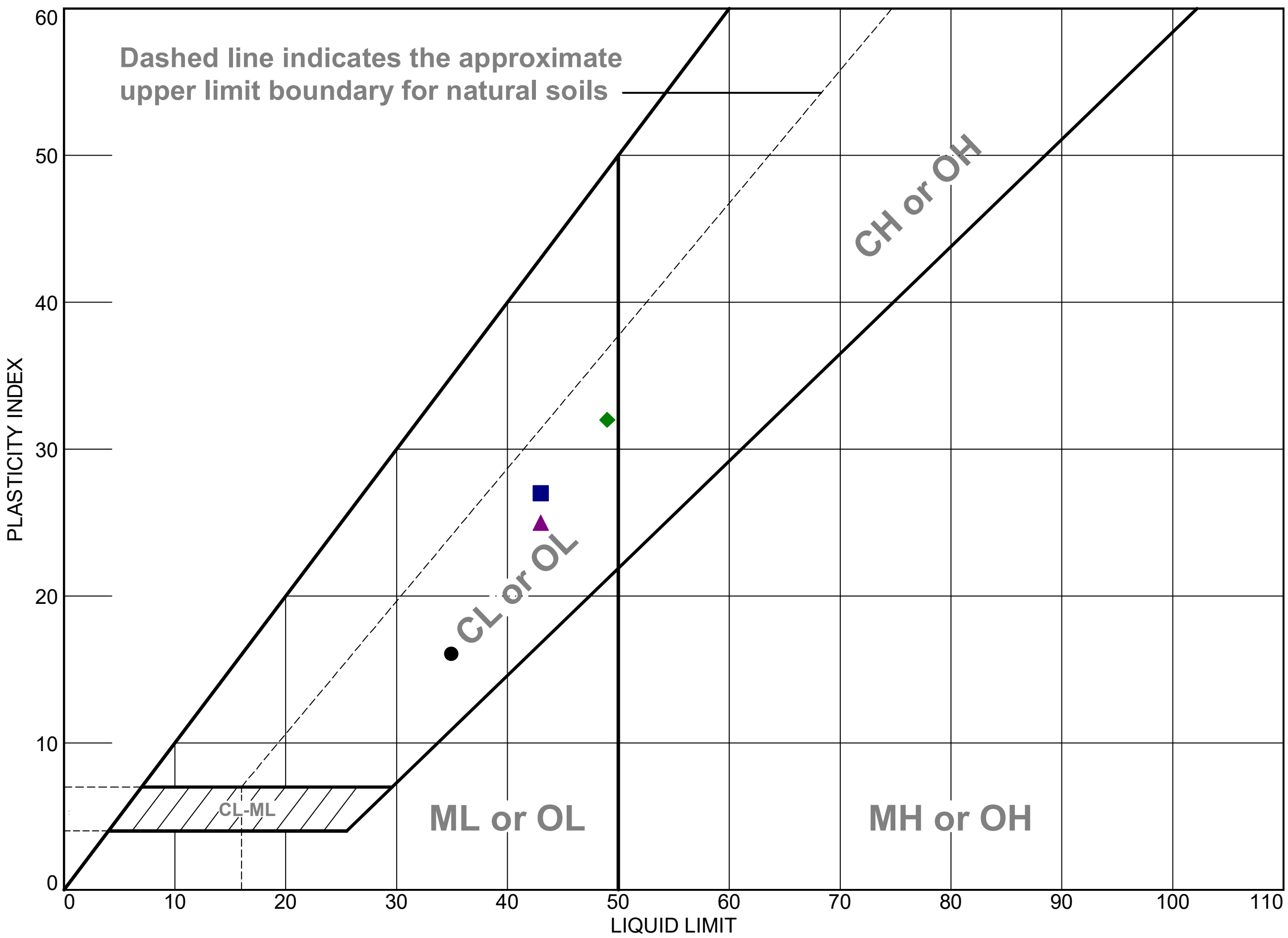
Source of Sample: P19070 Arroyo Road Bridge **Depth:** 40'
Sample Number: R-20-007, S6 **Date:**

Blackburn Consulting

W. Sacramento, CA

Client: WRECO
Project: WRECO Lab Testing
Project No: 3390.X **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CLAYEY SAND with GRAVEL, olive brown	35	19	16	35	20	SC
■	Lean CLAY, greenish gray	43	16	27		86	CL
▲	Lean CLAY, grayish green	43	18	25		94	CL
◆	Lean CLAY with SAND, yellowish brown	49	17	32	98	85	CL

Project No. 3390.X **Client:** WRECO

Project: WRECO Lab Testing

●**Source of Sample:** P19070 Arroyo Road Bridge **Depth:** 20' **Sample Number:** R-20-006, S5
■**Source of Sample:** P19070 Arroyo Road Bridge **Depth:** 38' **Sample Number:** R-20-006, S9
▲**Source of Sample:** P19070 Arroyo Road Bridge **Depth:** 53' **Sample Number:** R-20-006, S12
◆**Source of Sample:** P19070 Arroyo Road Bridge **Depth:** 40' **Sample Number:** R-20-007, S6

Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure

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

Date Reported 03/04/2020
Date Submitted 02/26/2020

To: Orion Adah
WRECO
1243 Alpine Rd. Ste 108
Walnut Creek, CA 94596

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

The reported analysis was requested for the following location:
Location : P19070 Site ID : R20-006 S-9@38.
Thank you for your business.

* For future reference to this analysis please use SUN # 81567-170315.

EVALUATION FOR SOIL CORROSION

Soil pH	7.67		
Minimum Resistivity	1.10 ohm-cm (x1000)		
Chloride	9.1 ppm	00.00091	%
Sulfate	13.8 ppm	00.00138	%

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

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

Date Reported 03/04/2020
Date Submitted 02/26/2020

To: Orion Adah
WRECO
1243 Alpine Rd. Ste 108
Walnut Creek, CA 94596

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

The reported analysis was requested for the following location:
Location : P19070 Site ID : R20-007 S-5@30.
Thank you for your business.

* For future reference to this analysis please use SUN # 81567-170316.

EVALUATION FOR SOIL CORROSION

Soil pH	7.51	
Minimum Resistivity	0.54 ohm-cm (x1000)	
Chloride	149.6 ppm	00.01496 %
Sulfate	27.1 ppm	00.00271 %

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

Appendix IV. Analyses and Calculations

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Appendix IV.1 Seismic Analysis

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Project Name: Arroyo Rd Bridge **Completed By:** BA on 2/28/2020
Project Number: P19070 **Checked By:** ____ on ____

Estimating Average Small Strain Shear Wave Velocity (VS30) for Top 100FT

Ref: Caltrans Geotechnical Services Design Manual Version 1.0 (Aug 2009)

Boring Number: R-20-006

1 m = 3.28084 ft

				COHESIONLESS							
				Using SPT (1) Sykora (1987) ER = 72.6				Using CPT (2) Mayne (2007)			
Layer	Method Used	Depth to Top (FT)	Depth To Bottom (FT)	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined	q_{t_ave} (MPa)	Effective Overburden	V_S (m/s)	V_S (m/s) Confined
1	1	0	5	10	12.1	207.097005	207.09701	0	0	1	1
2	1	5	10	30	36.3	284.799343	284.79934	0	0	1	1
3	1	10	15	60	72.6	348.207148	348.20715	0	0	1	1
4	1	15	20	33	39.93	292.780981	292.78098	0	0	1	1
5	1	20	23	100	121	403.806978	380	0	0	1	1
6	1	23	28	52	62.92	334.052548	334.05255	0	0	1	1
7	6	28	35	0	0	1	1	0	0	1	1
8	6	35	40	0	0	1	1	0	0	1	1
9	6	40	45	0	0	1	1	0	0	1	1
10	6	45	50	0	0	1	1	0	0	1	1
11	6	50	58	0	0	1	1	0	0	1	1
12	0	0	0	0	0	1	1	0	0	1	1
13	0	0	0	0	0	1	1	0	0	1	1
14	0	0	0	0	0	1	1	0	0	1	1
15	0	0	0	0	0	1	1	0	0	1	1
16	0	0	0	0	0	1	1	0	0	1	1
17	0	0	0	0	0	1	1	0	0	1	1
18	0	0	0	0	0	1	1	0	0	1	1
19	0	0	0	0	0	1	1	0	0	1	1
20	0	0	0	0	0	1	1	0	0	1	1

	COHESIVE										YOUNG SEDIMENTARY ROCK			
	Using SPT (3) Ohta and Goto (1978) ER = 72.6				Using S_u (4) Dickenson (1994)			Using CPT (5) Mayne and Rix (1995)			Using SPT (6) Imai & Tonouchi (1982) ER = 72.6			
Layer	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined	S_u (psf)	V_S (m/s)	V_S (m/s) Confined	q_{t_ave} (kPa)	V_S (m/s)	V_S (m/s) Confined	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined
1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
2	0	0	1	1	0	1	1	0	1	1	0	0	1	1
3	0	0	1	1	0	1	1	0	1	1	0	0	1	1
4	0	0	1	1	0	1	1	0	1	1	0	0	1	1
5	0	0	1	1	0	1	1	0	1	1	0	0	1	1
6	0	0	1	1	0	1	1	0	1	1	0	0	1	1
7	0	0	1	1	0	1	1	0	1	1	100	121	503.3094	503.3094
8	0	0	1	1	0	1	1	0	1	1	100	121	503.3094	503.3094
9	0	0	1	1	0	1	1	0	1	1	100	121	503.3094	503.3094
10	0	0	1	1	0	1	1	0	1	1	100	121	503.3094	503.3094
11	0	0	1	1	0	1	1	0	1	1	100	121	503.3094	503.3094
12	0	0	1	1	0	1	1	0	1	1	0	0	1	1
13	0	0	1	1	0	1	1	0	1	1	0	0	1	1
14	0	0	1	1	0	1	1	0	1	1	0	0	1	1
15	0	0	1	1	0	1	1	0	1	1	0	0	1	1
16	0	0	1	1	0	1	1	0	1	1	0	0	1	1
17	0	0	1	1	0	1	1	0	1	1	0	0	1	1
18	0	0	1	1	0	1	1	0	1	1	0	0	1	1
19	0	0	1	1	0	1	1	0	1	1	0	0	1	1
20	0	0	1	1	0	1	1	0	1	1	0	0	1	1

Enter Total Depth = 58

Method Numbering Key

- 0 = Layer Not Used
- 1 = Cohesionless Using SPT
- 2 = Cohesionless Using CPT
- 3 = Cohesive Using SPT
- 4 = Cohesive Using S_u
- 5 = Cohesive Using CPT
- 6 = Sedimentary Rock Using SPT

Layer	Depth to Top (FT)	Depth To Bottom (FT)	V_S (m/s)	V_S (ft/s)	D/V_S (sec)
1	0	5	207.09701	679.45212	0.00735887
2	5	10	284.79934	934.38105	0.005351136
3	10	15	348.20715	1142.4119	0.004376705
4	15	20	292.78098	960.56752	0.005205256
5	20	23	380	1246.7192	0.002406316
6	23	28	334.05255	1095.9729	0.004562156
7	28	35	503.30939	1651.2775	0.004239142
8	35	40	503.30939	1651.2775	0.003027959
9	40	45	503.30939	1651.2775	0.003027959
10	45	50	503.30939	1651.2775	0.003027959
11	50	58	503.30939	1651.2775	0.004844734
12	0	0	1	3.2808399	0
13	0	0	1	3.2808399	0
14	0	0	1	3.2808399	0
15	0	0	1	3.2808399	0
16	0	0	1	3.2808399	0
17	0	0	1	3.2808399	0
18	0	0	1	3.2808399	0
19	0	0	1	3.2808399	0
20	0	0	1	3.2808399	0

RESULTS Vs(d)

V_{Sd} = 1222.901 **ft/sec**

V_{Sd} = 372.74 **m/sec**

OR

Feet to meters conversion:
1-foot = 0.3048 meters

***ESTIMATING VS30 FOR SITES WITH SUBSURFACE INFO <100 ft (30 m)**

$V_{S30} = [1.45 - (0.015 * d)] * V_{S(d)}$

d = depth in *"meters"* to bottom of known soil column

$V_{S(d)}$ = Time averaged velocity (m/sec) for known soil column

Vs30 = **441.6317** m/sec

Other Rocks

Review Studies by:

1 Fumal (1978) - Correlated shear wave velociy to weathering, hardness, fracture spacing, and lithology based on data from 27 sites in San Francisco, CA.

2 Fumal and Tinsley (1985) - extended the 1978 study to include 84 sites in Los Angelas, CA

Note: In the absense of in-situ measurements of V_S, the V_{S30} for competent rocks in California should be limited to 760 m/sec

Project Name: Arroyo Rd Bridge **Completed By:** BA on 2/28/2020
Project Number: P19070 **Checked By:** on

Estimating Average Small Strain Shear Wave Velocity (VS30) for Top 100FT
Ref: Caltrans Geotechnical Services Design Manual Version 1.0 (Aug 2009)

Boring Number: R-20-007

1 m = 3.28084 ft

				COHESIONLESS								COHESIVE										YOUNG SEDIMENTARY ROCK			
				Using SPT (1) Sykora (1987), ER = 72.6				Using CPT (2) Mayne (2007)				Using SPT (3) Ohta and Goto (1978), ER = 72.6				Using S_u (4) Dickenson (1994)			Using CPT (5) Mayne and Rix (1995)			Using SPT (6) Imai & Tonouchi (1982), ER = 72.6			
Layer	Method Used	Depth to Top (FT)	Depth To Bottom (FT)	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined	q_{t_ave} (MPa)	Effective Overburden	V_S (m/s)	V_S (m/s) Confined	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined	S_u (psf)	V_S (m/s)	V_S (m/s) Confined	q_{t_ave} (kPa)	V_S (m/s)	V_S (m/s) Confined	N_{ave}	N_{60}	V_S (m/s)	V_S (m/s) Confined
1	1	0	5	10	12.1	207.097005	207.09701	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
2	1	5	10	14	16.94	228.323677	228.32368	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
3	1	10	15	59	71.39	346.514093	346.51409	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
4	1	20	30	38	45.98	305.007882	305.00788	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
5	3	30	40	0	0	1	1	0	0	1	1	28	33.88	280.8607	280.860671	0	1	1	0	1	1	0	0	1	1
6	3	40	50	0	0	1	1	0	0	1	1	47	56.87	333.7305	310	0	1	1	0	1	1	0	0	1	1
7	3	50	70	0	0	1	1	0	0	1	1	59	71.39	359.9822	310	0	1	1	0	1	1	0	0	1	1
8	3	70	71.5	0	0	1	1	0	0	1	1	43	52.03	323.9905	310	0	1	1	0	1	1	0	0	1	1
9	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
10	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
11	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
12	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
13	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
14	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
15	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
16	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
17	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
18	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
19	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1
20	0	0	0	0	0	1	1	0	0	1	1	0	0	1	1	0	1	1	0	1	1	0	0	1	1

Enter Total Depth = 71.5

Method Numbering Key

- **0 = Layer Not Used**
- **1 = Cohesionless Using SPT**
- **2 = Cohesionless Using CPT**
- **3 = Cohesive Using SPT**
- **4 = Cohesive Using S_u**
- **5 = Cohesive Using CPT**
- **6 = Sedimentary Rock Using SPT**

Layer	Depth to Top (FT)	Depth To Bottom (FT)	V_S (m/s)	V_S (ft/s)	D/V_S (sec)
1	0	5	207.09701	679.45212	0.00735887
2	5	10	228.32368	749.09343	0.006674735
3	10	15	346.51409	1136.8573	0.00439809
4	20	30	305.00788	1000.682	0.009993184
5	30	40	280.86067	921.4589	0.010852356
6	40	50	310	1017.0604	0.009832258
7	50	70	310	1017.0604	0.019664516
8	70	71.5	310	1017.0604	0.001474839
9	0	0	1	3.2808399	0
10	0	0	1	3.2808399	0
11	0	0	1	3.2808399	0
12	0	0	1	3.2808399	0
13	0	0	1	3.2808399	0
14	0	0	1	3.2808399	0
15	0	0	1	3.2808399	0
16	0	0	1	3.2808399	0
17	0	0	1	3.2808399	0
18	0	0	1	3.2808399	0
19	0	0	1	3.2808399	0
20	0	0	1	3.2808399	0

RESULTS Vs(d)

V_{Sd} = 1017.81 **ft/sec**

V_{Sd} = 310.23 **m/sec**

OR

Feet to meters conversion:	
1-foot = 0.3048 meters	

***ESTIMATING VS30 FOR SITES WITH SUBSURFACE INFO <100 ft (30 m)**

V_{S30} = [1.45 - (0.015 * d)]* $V_{S(d)}$

d = depth in *"meters"* to bottom of known soil column

$V_{S(d)}$ = Time averaged velocity (m/sec) for known soil column

Vs30 = 348.4183 m/sec

Other Rocks

Review Studies by

1. Fumal (1978) - Correlated shear wave velociy to weathering, hardness, fracture spacing, and lithology based on data from 27 sites in San Francisco, CA.
2. Fumal and Tinsley (1985) - extended the 1978 study to include 84 sites in Los Angelas, CA

Note: In the absense of in-situ measurements of V_S, the V_{S30} for competent rocks in California should be limited to 760 m/sec

wreco

ARS Online V3.0.2

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 37.63781 **Longitude:** -121.76364 **Vs30 (m/s):** 348 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.64	0.68	1	1	1	0.64	0.68
0.10	1.21	1.24	1	1	1	1.21	1.24
0.20	1.51	1.63	1	1	1	1.51	1.63
0.30	1.44	1.69	1	1	1	1.44	1.69
0.50	1.17	1.46	1	1	1	1.17	1.46
0.75	0.9	1.1	1	1	1.09	0.98	1.2
1.0	0.69	0.85	1	1	1.19	0.82	1.01
2.0	0.32	0.4	1	1	1.19	0.38	0.47
3.0	0.19	0.24	1	1	1.19	0.23	0.29
4.0	0.13	0.17	1	1	1.19	0.16	0.2
5.0	0.11	0.13	1	1	1.19	0.13	0.15

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	6.71
mean site-source distance (km, for Sa at 1s)	15.7

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Appendix IV.2 Spread Footing Analysis

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Project No: P19070

CALCULATION SHEET

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007

Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: Clayey Sand/Sandy Clay			**Soil Type: Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or buoyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or buoyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, $q_n = \gamma_1 D N_q + 1/2 \gamma_2 B' N_{\gamma,}$ for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q :	73.9	Unit Weight (Footing width term), N_γ :	109.4
for φ =	40	for φ =	40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = $\gamma_1 D N_q + 1/2 \gamma_2 B' N_{\gamma,}$ for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	5.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 16.60 kips per square foot

IF 16.60 > 4.82 SAY OK

Service Limit State 1 Bearing Capacity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	5.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 16.60 kips per square foot

IF 16.60 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	5.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 36.88 kips per square foot

IF 36.88 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

CALCULATION SHEET

Project No: P19070

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007
Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: Clayey Sand/Sandy Clay			**Soil Type: Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or bouyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or bouyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, $q_n = \gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$, for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q :	73.9	Unit Weight (Footing width term), N_γ :	109.4
for φ =	40	for φ =	40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = $\gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$, for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	6.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 18.32 kips per square foot

IF 18.32 > 4.82 SAY OK

Service Limit State 1 Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	6.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 18.32 kips per square foot

IF 18.32 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	6.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 40.71 kips per square foot

IF 40.71 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

WRECO

Project No: P19070

CALCULATION SHEET

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007

Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: Clayey Sand/Sandy Clay			**Soil Type: Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or bouyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or bouyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, $q_n = \gamma_1 D N_q + 1/2\ \gamma_2 B' N_\gamma$, for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q :	73.9	Unit Weight (Footing width term), N_γ :	109.4
for φ =	40	for φ =	40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = $\gamma_1 D N_q + 1/2\ \gamma_2 B' N_\gamma$, for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	7.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 20.04 kips per square foot

IF 20.04 > 4.82 SAY OK

Service Limit State 1 Bearing Capacity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	7.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 20.04 kips per square foot

IF 20.04 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	7.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 44.54 kips per square foot

IF 44.54 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

CALCULATION SHEET

Sheet No.: 4 OF 6

Project No: P19070

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007

Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: Clayey Sand/Sandy Clay			**Soil Type: Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or bouyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or bouyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, $q_n = \gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$ for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q : 73.9		Unit Weight (Footing width term), N_γ : 109.4
for φ = 40		for φ = 40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = $\gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$ for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	8.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 21.77 kips per square foot

IF 21.77 > 4.82 SAY OK

Service Limit State 1 Bearing Capacity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	8.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 21.77 kips per square foot

IF 21.77 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	8.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 48.37 kips per square foot

IF 48.37 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

Project No: P19070

CALCULATION SHEET

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007
Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: **Clayey Sand/Sandy Clay**			**Soil Type:** **Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or bouyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or bouyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, $q_n = \gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$, for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q :	73.9	Unit Weight (Footing width term), N_γ :	109.4
for φ =	40	for φ =	40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = $\gamma_1 D N_q + 1/2\, \gamma_2 B' N_\gamma$, for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	9.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 23.49 kips per square foot

IF 23.49 > 4.82 SAY OK

Service Limit State 1 Bearing Capacity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	9.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 23.49 kips per square foot

IF 23.49 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	9.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 52.20 kips per square foot

IF 52.20 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

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Sheet No.: 6 OF 6

Project No: P19070

CALCULATION SHEET

Project Titile : Arroyo Road Bridge
Subject : Abuts
Analysis: Bearing Capacity Analysis Cohesionless for Soils

By: FPT
Check by:
Date: 6/10/2020

Soil Boring: R-20-006 & R-20-007
Bottom of Footing Elevation: 499.0 ft
Rough Grade: 502.0 ft

Foundation Soil Properties above the bottom of footing:			Foundation Soil Properties below bottom of footing:		
Soil Type: Clayey Sand/Sandy Clay			**Soil Type: Clayey Sand/Sandy Clay**		
Depth of Embedment, D :	3	feet			
Unit Weight of Soil (total or buoyant), γ_1	80	pounds per cubic foot	Unit Weight of Soil (total or buoyant), γ_2	70	pounds per cubic foot
Internal Angle of Friction , φ	40	degrees	Internal Angle of Friction , φ	40	degrees
Cohesion, c	0	pounds per square foot	Cohesion, c	0	pounds per square foot

NOMINAL BEARING CAPACITY, q_n = γ_1 D N_q+1/2 γ_2 B' N_{γ}, for cohesionless soils

Input Parameters :

Surcharge (Embedment) Term, N_q :	73.9	Unit Weight (Footing width term), N_γ :	109.4
for φ =	40	for φ =	40

Note: Bearing capacity factors from AASHTO, 2007, LRFD Table 10.6.3.1.2a-1

FACTORED BEARING CAPACITY EQUATION = γ_1 D N_q+1/2 γ_2 B' N_{γ}, for cohesionless soils

Strength Limit State Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.8	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	10.0	feet
Strength Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 25.21 kips per square foot

IF 25.21 > 4.82 SAY OK

Service Limit State 1 Bearing Capacity Check

Gross Bearing Pressure (from Designer) :	3.1	kips per sq.foot
eccentricity,e:		feet
Effective Footing Width, B':	10.0	feet
Service Limit Phi Factor for Bearing Capacity, Φ:	0.45	SPT, LRFD Article 10.5.5.2.2-1

Factored Bearing Capacity = 25.21 kips per square foot

IF 25.21 > 3.12 SAY OK

Extreme I Event Bearing Capactity Check

Gross Bearing Pressure (from Designer) :	4.2	kips per square foot
eccentricity, e:		ft
Effective Footing width (from Designer), B' :	10.0	feet
Extreme Event Phi Factor for Bearing Capacity, F:	1.00	LRFD Article 10.5.5.3.3

Factored Bearing Capacity = 56.03 kips per square foot

IF 56.03 > 4.22 SAY OK

Maximum allowable concrete bearing stress = 0.3 f'c per Section 10.6.2.6.2 of reference AASHTO, 2007.

References:

(a) NAVFAC, 1986. Ultimate Bearing Capacity of Shallow Footings with Concentric Loads, 7.2-131. Naval Facilites Engineering Command, Foundations &

(b) AASHTO, 2007. AASHTO LRFD Bridge Design Specifications, with Caltrans Amendments, 4th Edition, 2007

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CLIENT:Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)		Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
				(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)				
5	8.4	3.0		0.195	8.205	5	0.520	0.897

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	8	0.520
0	8	32	0.065		0.000		0.000

INFLUENCE VALUES & BOUNDARY CONDITIONS:

3 FT = D, FIRST LAYER MUST START AT DEPTH D
8 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
23 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	8	5	5.5	465	0.549	0.00590
8	23	7.8	15.5	465	0.449	0.00753
23	23	0	23	3000	0.000	0.00000
SUMMATION, Iz*D/Es =						0.01342

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es		SETTLEMENT (FT.)	SETTLEMENT (IN.)
8.205	0.99	50	1.54	0.0134		0.1676	2.01

CLIENT:Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)		Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
				(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)				
6	6.7	3.0		0.195	6.505	6	0.585	0.833

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	9	0.520
0	8	32	0.065		0.000		0.065

INFLUENCE VALUES & BOUNDARY CONDITIONS:
3 FT = D, FIRST LAYER MUST START AT DEPTH D
9 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
27 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	9	6	6	465	0.517	0.00667
9	20	7.8	14.5	465	0.579	0.00971
20	27	7	23.5	3000	0.162	0.00038
SUMMATION, Iz*D/Es =						0.01675

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es		SETTLEMENT (FT.)	SETTLEMENT (IN.)
6.505	0.99	50	1.54	0.0168		0.1653	1.98

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CLIENT:Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)	Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
7	6.1	3.0	0.195	5.905	7	0.650	0.801

(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	10	0.520
0	8	32	0.065		0.000		0.130

INFLUENCE VALUES & BOUNDARY CONDITIONS:

3 FT = D, FIRST LAYER MUST START AT DEPTH D
10 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
31 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	10	7	6.5	465	0.501	0.00754
10	20	7.8	15	465	0.611	0.01024
20	31	11	25.5	3000	0.210	0.00077
SUMMATION, Iz*D/Es =						0.01855

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es	SETTLEMENT (FT.)	SETTLEMENT (IN.)
5.905	0.98	50	1.54	0.0185	0.1659	**1.99**

WRECO

CLIENT: Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)		Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
				(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)				
8	5.6	3.0		0.195	5.405	8	0.715	0.775

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	11	0.520
0	8	32	0.065		0.000		0.195

INFLUENCE VALUES & BOUNDARY CONDITIONS:

3 FT = D, FIRST LAYER MUST START AT DEPTH D
11 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
35 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	11	8	7	465	0.487	0.00839
11	20	7.8	15.5	465	0.630	0.01056
20	35	15	27.5	3000	0.242	0.00121
SUMMATION, Iz*D/Es =						0.02016

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es		SETTLEMENT (FT.)	SETTLEMENT (IN.)
5.405	0.98	50	1.54	0.0202		0.1648	**1.98**

WRECO

CLIENT:Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)		Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
				(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)				
9	5.3	3.0		0.195	5.105	9	0.780	0.756

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	12	0.520
0	8	50	0.065		0.000		0.260

INFLUENCE VALUES &
BOUNDARY CONDITIONS:

3 FT = D, FIRST LAYER MUST START AT DEPTH D
12 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
39 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	12	9	7.5	465	0.478	0.00925
12	20	7.8	16	465	0.644	0.01080
20	39	19	29.5	3000	0.266	0.00168
SUMMATION, Iz*D/Es =						0.02173

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es		SETTLEMENT (FT.)	SETTLEMENT (IN.)
5.105	0.98	50	1.54	0.0217		0.1676	2.01

WRECO

CLIENT:Wood Rodgers
Arroyo Road Bridge
PROJECT NO.: P19070
TITLE: Settlement at Elev. 499

STRUCTURE NO.: Abuts

COMPUTED BY: FPT
DATE: 6/10/2020
CHECKED BY: ________________
DATE: ______________________

SETTLEMENT OF STRIP FOOTINGS ON GRANULAR SOILS

(SCHMERTMANN, FHWA-TS-78-209, PGS 49-54)

EQUATIONS: Izp max = 0.5 + 0.1*(DELTA P / SIGMA V')^0.5 ; DELTA P = P - Po'

B, FOOTING WIDTH (FT.)	P, LOAD (KSF)	DEPTH TO FOOTING BASE (FT.)		Po ' (KSF)	DELTA P (KSF)	B (FT.)	SIGMA V ' (KSF)	Izp (MAX)
				(VALUES FOR Po' AND SIGMA V ' COMPUTED BELOW)				
10	4.9	3.0		0.195	4.705	10	0.845	0.736

NOTE: STRATA DEPTH START AT GROUND SURFACE AND EXTEND TO D + 4B
ONE STRATA BREAK AT WATER TABLE, USE BOUYANT UNIT WEIGHTS BELOW WATER TABLE

WATER TABLE DEPTH (FT.)	STRATA DEPTH FROM (FT.)	STRATA DEPTH TO (FT.)	UNIT WEIGHT (KCF)	P ZERO DEPTH (FT.)	P ZERO (KSF)	Izp DEPTH (FT.)	SIGMA V (KSF)
0	0	8	0.065	3	0.195	13	0.520
0	8	50	0.065		0.000		0.325

INFLUENCE VALUES & BOUNDARY CONDITIONS:
3 FT = D, FIRST LAYER MUST START AT DEPTH D
13 FT = D+B, ONE LAYER BOUNDARY MUST OCCUR AT Izp @ D+B
43 FT = D+4B, BOTTOM OF LAST LAYER MUST END AT D + 4B

LAYER DEPTH FROM (FT.)	LAYER DEPTH TO (FT.)	LAYER THICKNESS (FT.)	LAYER MID-THICK DEPTH, FT	E (KSF)	Iz	Iz*D/Es
-	-	-	-	-	-	-
3	13	10	8	465	0.468	0.01006
13	20	7.8	16.5	465	0.650	0.01090
20	43	23	31.5	3000	0.282	0.00216
SUMMATION, Iz*D/Es =						0.02313

SETTLEMENT COMPUTATION:

EQUATIONS:
C1 = 1.0-0.5*(P zero/DELTA P)
C2 = 1.0+0.2*LOG(TIME,YRS/0.1)
S = DELTA P*C1*C2*(SUM(Iz*D/Es)

DELTA P	C1	TIME (YRS)	C2	Iz*D/Es		SETTLEMENT (FT.)	SETTLEMENT (IN.)
4.705	0.98	50	1.54	0.0231		0.1641	1.97

Appendix IV.3 Pile Capacity Analysis

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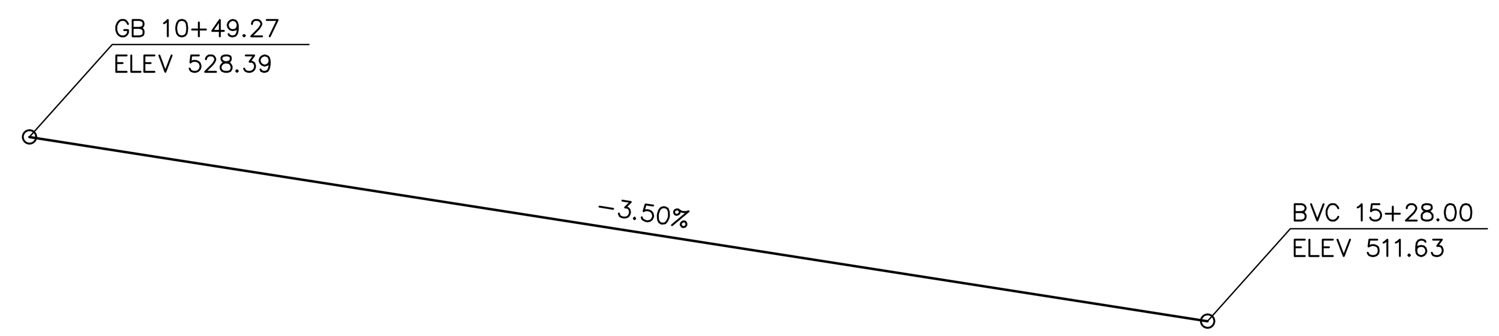
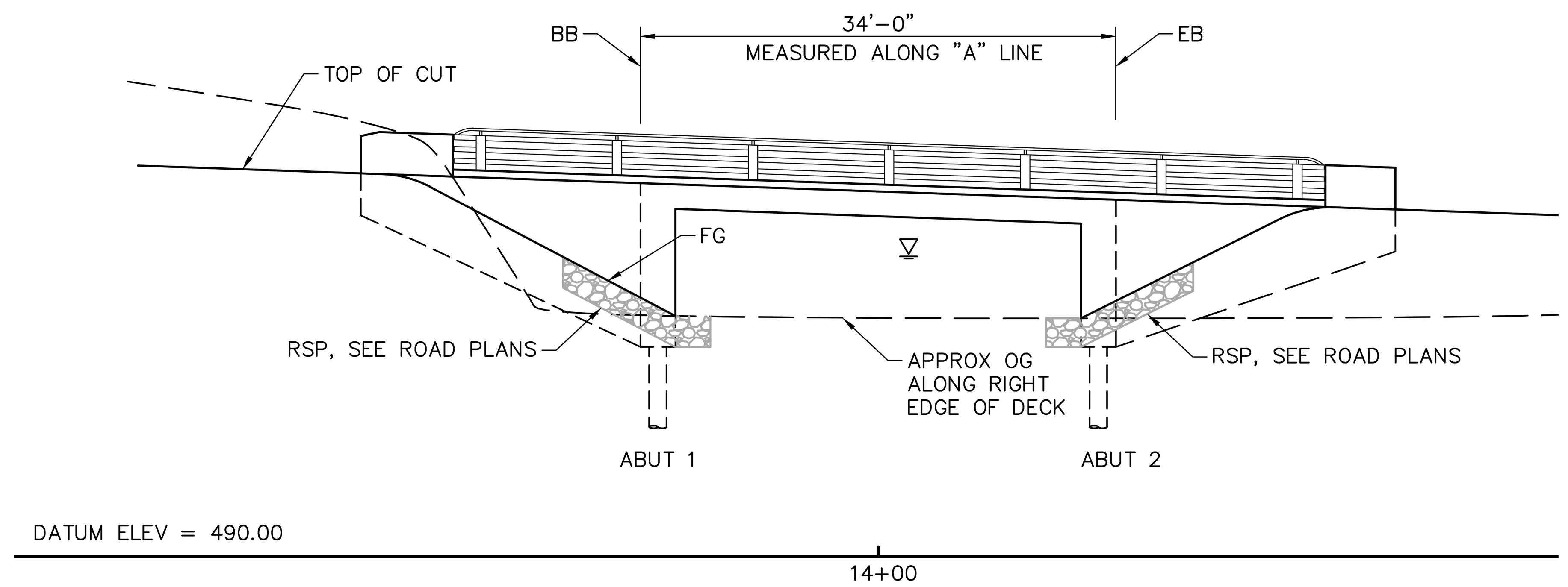


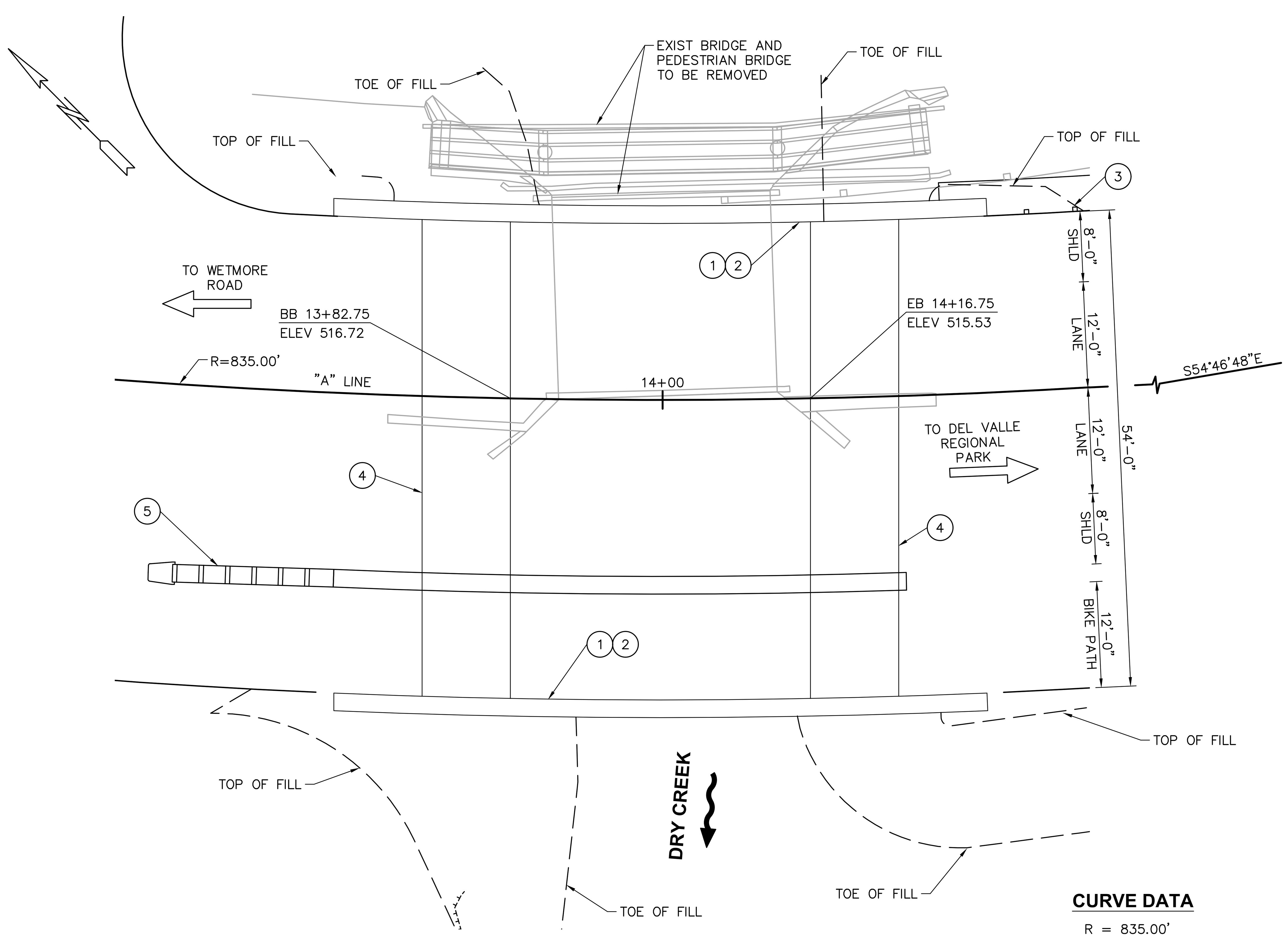
PROFILE GRADE

NTS



ELEVATION

SCALE: 1"=10'

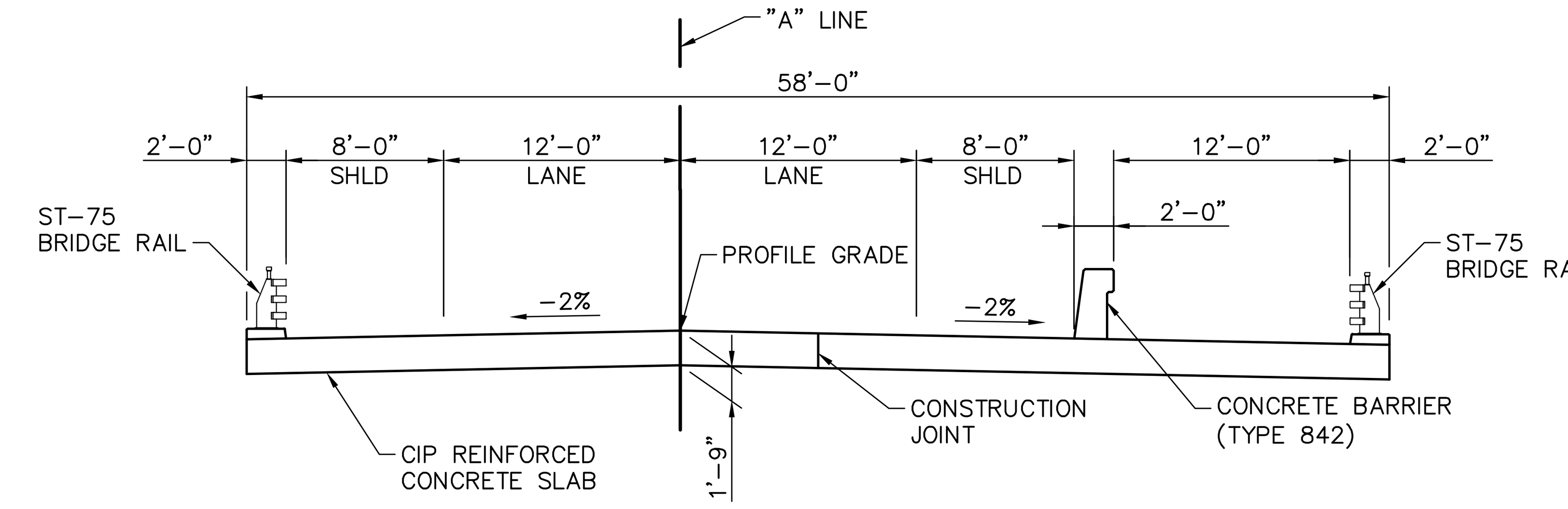


PLAN

SCALE: 1"=10'

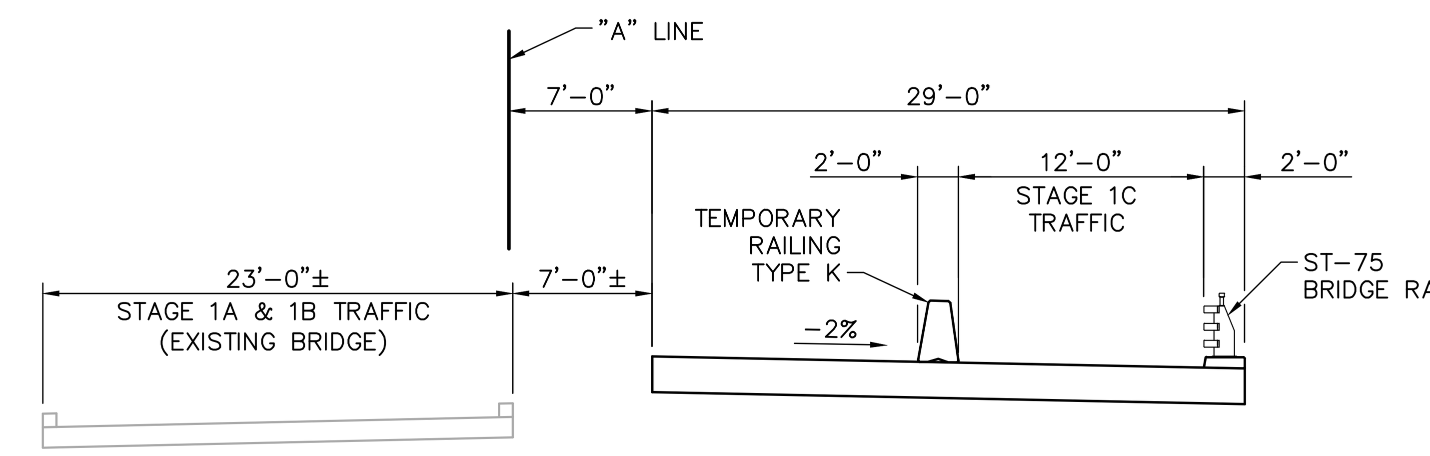
CURVE DATA

R = 835.00'
Δ = 31°24'15"
T = 234.74'
L = 457.67'



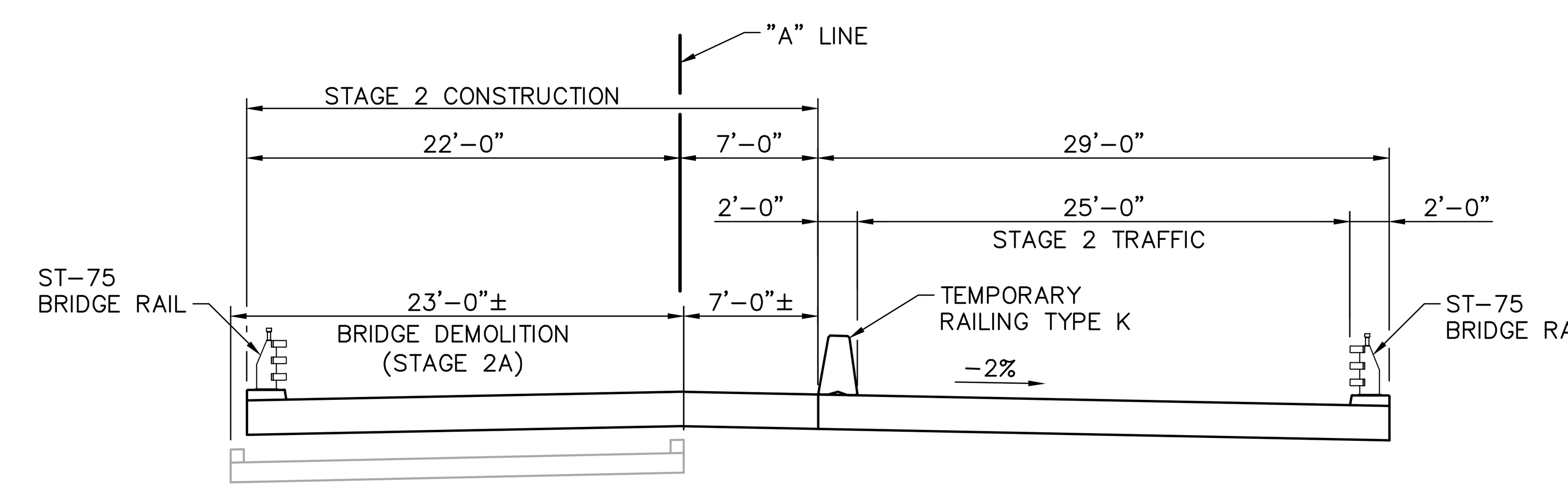
TYPICAL SECTION

SCALE: 3/8"=1'-0"



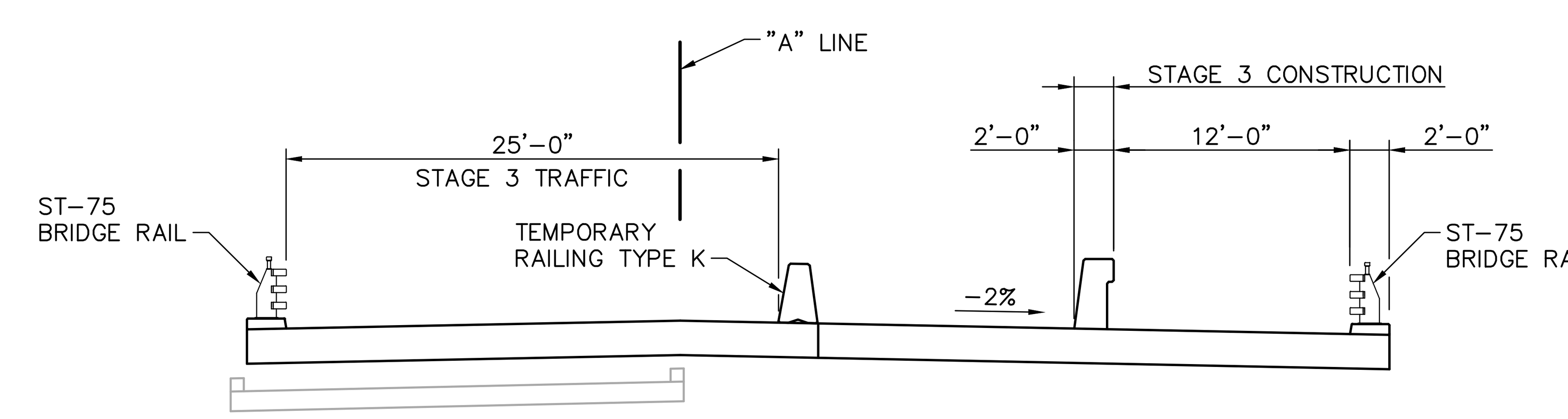
STAGE 1 CONSTRUCTION

SCALE: 3/8"=1'-0"



STAGE 2 CONSTRUCTION

SCALE: 3/8"=1'-0"



STAGE 3 CONSTRUCTION

SCALE: 3/8"=1'-0"

LEGEND

∇	FOR HYDROLOGIC/HYDRAULIC DATA SUMMARY, SEE "FOUNDATION PLAN" SHEET
	INDICATES DIRECTION OF FLOW
	INDICATES DIRECTION OF TRAFFIC

NOTES

1. PAINT BRIDGE NO. "XXXXXXX"
2. PAINT "ARROYO ROAD BRIDGE"
3. MIDWEST GUARDRAIL SYSTEM, SEE "ROAD PLANS"
4. STRUCTURE APPROACH TYPE EQ (10)
5. CRASH CUSHION, SEE "ROAD PLANS"

FOR REDUCED ENGLISH PLANS
ORIGINAL SCALE IS IN INCHES
0 1 2 3

REVIEWED BY:	DATE:		REVIEWED BY:	DATE:
CONSTRUCTION		SURVEY		
MAINTENANCE		TRAFFIC		
REAL ESTATE		ENVIRONMENTAL		

C:\projects\1030(Wood Rodgers)\010 (Alameda County)\01 (Arroyo Rd Br)\8781.002 1-GP.dwg 7-27-20 01:30:40 PM AI

REVISIONS

NO.	DESCRIPTION	BY	DATE	APPV'D

APPROVED
APPROVAL DATE:

WOOD RODGERS
BUILDING RELATIONSHIPS ONE PROJECT AT A TIME
3301 C ST. BLDG. 100-B TEL 916.341.7760
SACRAMENTO, CA 95816 FAX 916.341.7767

DRAWN: A. SANCHEZ
DESIGNED: E. WESTON
CHECKED: S. RANDALL
APPROVED: C. HODGE

COUNTY OF ALAMEDA ☆ PUBLIC WORKS AGENCY

REVIEWED: | REVIEWED: | APPROVAL RECOMMENDED:

BRIDGE REPLACEMENT PROJECT ON
ARROYO ROAD
AT DRY CREEK

GENERAL PLAN

DATE (ADV DATE)	SCALE AS NOTED
WORK ORDER NO. RXXXXX	
SPECIFICATION NO. ####	
SHEET NO. OF ##	
FILE NO. U-XXX	

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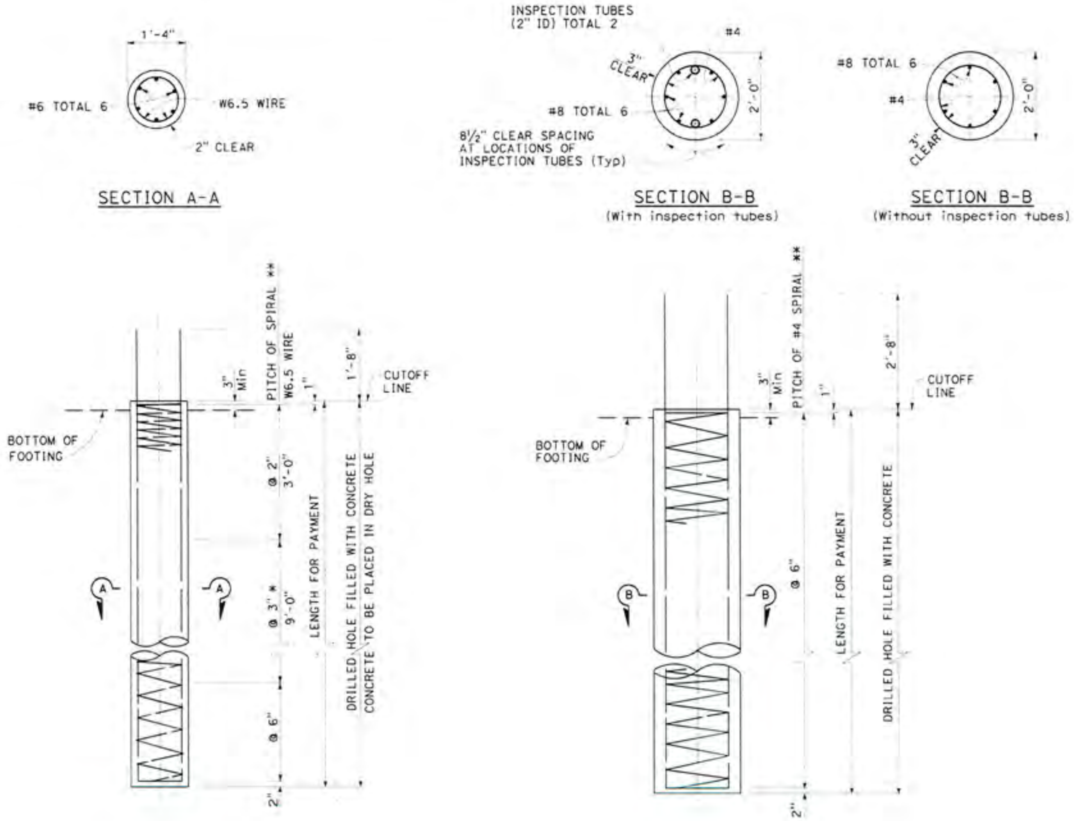

ELEVATION
90 kip AND 140 kip
DESIGN CAPACITY

ELEVATION
200 kip
DESIGN CAPACITY

* @ 2" at option of Contractor

** Extend at 2" pitch to top of anchor piles and load test piles. For longitudinal reinforcement for anchor piles and load test piles, see "Load Test Pile Details (2)", Standard Plan B2-10.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

REGISTERED CIVIL ENGINEER

May 31, 2018

PLANS APPROVAL DATE

Amir M. Malek
No. C62397
Exp. 9-30-19

NOTES:

1. Reinforcement extending into footing shall be hooked as required to provide clearance to top of footing.
2. Piles shall be extended only in accordance with details shown on the Project Plans.

DESIGN NOTES:

REINFORCED CONCRETE

f_y = 60,000 psi

f'_c = 4,000 psi

DESIGN CAPACITY

90 kip and 140 kip PILE

COMPRESSION:

140 kip (Service state)
280 kip (Nominal axial structural resistance)

TENSION:

56 kip (Service state)
140 kip (Nominal axial structural resistance)

200 kip PILE

COMPRESSION:

200 kip (Service state)
400 kip (Nominal axial structural resistance)

TENSION:

80 kip (Service state)
200 kip (Nominal axial structural resistance)

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

16" AND 24"
CAST-IN-DRILLED-HOLE
CONCRETE PILE

NO SCALE

B2-3

Return to Table of Contents

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```
==========================================================================

                 SHAFT for Windows, Version 2017.8.2

                  Serial Number :  158117577

               VERTICALLY LOADED DRILLED SHAFT ANALYSIS
                (c) Copyright ENSOFT, Inc., 1987-2017
                     All Rights Reserved

==========================================================================


Path to file locations      : G:\Projects\Y2019\P19070 Arroyo Rd Br Dry Cr\Calculations\SHAFT\
Name of input data file     : Arroyo 24 in.sf8d
Name of output file         : Arroyo 24 in.sf8o
Name of plot output file    : Arroyo 24 in.sf8p
Name of runtime file        : Arroyo 24 in.sf8r


--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  June 10, 2020     Time:  15:41:56

Arroyo 24" CIDH cutoff el. 503 scour el 499


TOTAL LOAD =     140.0 TONS
------------


NUMBER OF LAYERS =    3
------------------


WATER TABLE DEPTH =     100.0 FT.
-------------------




SOIL INFORMATION
---------------

LAYER NO 1----SAND

  AT THE TOP

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.120E+01  (*)
  INTERNAL FRICTION ANGLE, DEG.                     = 0.410E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.650E+02
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.000E+00

  AT THE BOTTOM

  SIDE FRICTION PROCEDURE, BETA METHOD
  SKIN FRICTION COEFFICIENT- BETA                   = 0.853E+00  (*)
  INTERNAL FRICTION ANGLE, DEG.                     = 0.410E+02
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST     = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                        = 0.650E+02
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT          = 0.100E+11
  DEPTH, FT                                         = 0.230E+02

  LRFD RESISTANCE FACTOR (SIDE FRICTION)            = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)           = 0.100E-03


LAYER NO 2----CLAY
```

```
  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.511E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.650E+02
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.230E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.511E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.650E+02
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.330E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.100E-03


LAYER NO 3----CLAY

  AT THE TOP

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.511E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.330E+02


  AT THE BOTTOM

  STRENGTH REDUCTION FACTOR-ALPHA                    = 0.511E+00  (*)
  END BEARING COEFFICIENT-Nc                         = 0.900E+01  (*)
  UNDRAINED SHEAR STRENGTH, LB/SQ FT                 = 0.400E+04
  BLOWS PER FOOT FROM STANDARD PENETRATION TEST      = 0.000E+00
  SOIL UNIT WEIGHT, LB/CU FT                         = 0.125E+03
  MAXIMUM LOAD TRANSFER FOR SOIL, LB/SQ FT           = 0.100E+11
  DEPTH, FT                                          = 0.800E+02


  LRFD RESISTANCE FACTOR (SIDE FRICTION)             = 0.700E+00
  LRFD RESISTANCE FACTOR (TIP RESISTANCE)            = 0.100E-03


(*) ESTIMATED BY THE PROGRAM BASED ON OTHER PARAMETERS


INPUT DRILLED SHAFT INFORMATION
------------------------------

 MINIMUM SHAFT DIAMETER    =    2.000  FT.
 MAXIMUM SHAFT DIAMETER    =    2.000  FT.
 RATIO BASE/SHAFT DIAMETER =    0.000  FT.
 ANGLE OF BELL             =    0.000  DEG.
 IGNORED TOP PORTION       =    5.000  FT.
 IGNORED BOTTOM PORTION    =    0.000  FT.
 ELASTIC MODULUS, Ec       = 0.340E+07 LB/SQ IN
```

COMPUTATION RESULTS

- CASE ANALYZED : 1
  VARIATION LENGTH : 1
  VARIATION DIAMETER : 1

DRILLED SHAFT INFORMATION

DIAMETER OF STEM	= 2.000	FT.
DIAMETER OF BASE	= 2.000	FT.
END OF STEM TO BASE	= 0.000	FT.
ANGLE OF BELL	= 0.000	DEG.
IGNORED TOP PORTION	= 5.000	FT.
IGNORED BOTTOM PORTION	= 0.000	FT.
AREA OF ONE PERCENT STEEL	= 4.524	SQ.IN.
ELASTIC MODULUS, Ec	= 0.340E+07	LB/SQ IN
VOLUME OF UNDERREAM	= 0.000	CU.YDS.

PREDICTED RESULTS

QS = ULTIMATE SIDE RESISTANCE;
QB = ULTIMATE BASE RESISTANCE;
WT = WEIGHT OF DRILLED SHAFT (UPLIFT CAPACITY ONLY);
QU = TOTAL ULTIMATE RESISTANCE;
LRFD QS = TOTAL SIDE FRICTION USING LRFD RESISTANCE FACTOR TO THE ULTIMATE SIDE RESISTANCE;
LRFD QB = TOTAL BASE BEARING USING LRFD RESISTANCE FACTOR TO THE ULTIMATE BASE RESISTANCE
LRFD QU = TOTAL CAPACITY WITH LRFD RESISTANCE FACTOR.

LENGTH (FT)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	LRFD QS (TONS)	LRFD QB (TONS)	LRFD QU (TONS)
6.0	0.70	1.31	43.09	44.40	0.92	0.00	0.92
7.0	0.81	2.83	47.80	50.63	1.98	0.00	1.99
8.0	0.93	4.54	52.52	57.06	3.18	0.01	3.19
9.0	1.05	6.44	57.23	63.67	4.51	0.01	4.52
10.0	1.16	8.53	61.94	70.47	5.97	0.01	5.97
11.0	1.28	10.78	66.66	77.44	7.55	0.01	7.55
12.0	1.40	13.21	71.37	84.58	9.25	0.01	9.25
13.0	1.51	15.79	76.08	91.88	11.06	0.01	11.06
14.0	1.63	18.54	80.79	99.33	12.98	0.01	12.98
15.0	1.75	21.43	85.51	106.94	15.00	0.01	15.01
16.0	1.86	24.47	90.22	114.69	17.13	0.01	17.14
17.0	1.98	27.65	93.25	120.90	19.35	0.01	19.36
18.0	2.09	30.96	94.26	125.22	21.67	0.01	21.68
19.0	2.21	34.41	94.26	128.67	24.08	0.01	24.09
20.0	2.33	37.98	80.79	118.77	26.58	0.01	26.59
21.0	2.44	41.67	64.64	106.30	29.17	0.01	29.17
22.0	2.56	45.47	56.56	102.03	31.83	0.01	31.84
23.0	2.68	49.39	56.56	105.95	34.57	0.01	34.58
24.0	2.79	55.81	56.56	112.37	39.07	0.01	39.07
25.0	2.91	62.23	56.56	118.79	43.56	0.01	43.57
26.0	3.03	68.66	56.56	125.21	48.06	0.01	48.07
27.0	3.14	75.08	56.56	131.64	52.56	0.01	52.56
28.0	3.26	81.50	56.56	138.06	57.05	0.01	57.06
29.0	3.37	87.93	56.56	144.48	61.55	0.01	61.55
30.0	3.49	94.35	56.56	150.90	66.04	0.01	66.05
31.0	3.61	100.77	56.56	157.33	70.54	0.01	70.55
32.0	3.72	107.19	56.56	163.75	75.04	0.01	75.04
33.0	3.84	113.62	56.56	170.17	79.53	0.01	79.54
34.0	3.96	120.04	56.56	176.60	84.03	0.01	84.03

35.0	4.07	126.46	56.56	183.02	88.52	0.01	88.53
36.0	4.19	132.88	56.56	189.44	93.02	0.01	93.02
37.0	4.31	139.31	56.56	195.86	97.52	0.01	97.52
38.0	4.42	145.73	56.56	202.29	102.01	0.01	102.02
39.0	4.54	152.15	56.56	208.71	106.51	0.01	106.51
40.0	4.65	158.58	56.56	215.13	111.00	0.01	111.01
41.0	4.77	165.00	56.56	221.55	115.50	0.01	115.50
42.0	4.89	171.42	56.56	227.98	119.99	0.01	120.00
43.0	5.00	177.84	56.56	234.40	124.49	0.01	124.50
44.0	5.12	184.27	56.56	240.82	128.99	0.01	128.99
45.0	5.24	190.69	56.56	247.24	133.48	0.01	133.49
46.0	5.35	197.11	56.56	253.67	137.98	0.01	137.98
47.0	5.47	203.53	56.56	260.09	142.47	0.01	142.48

AXIAL LOAD VS SETTLEMENT CURVES

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.5696E-01	0.3600E-04	0.1862E-02	0.1000E-04
0.2848E+00	0.1800E-03	0.9308E-02	0.5000E-04
0.5696E+00	0.3600E-03	0.1862E-01	0.1000E-03
0.2898E+02	0.1819E-01	0.9308E+00	0.5000E-02
0.4347E+02	0.2729E-01	0.1396E+01	0.7500E-02
0.5772E+02	0.3635E-01	0.1862E+01	0.1000E-01
0.1213E+03	0.8234E-01	0.4654E+01	0.2500E-01
0.1692E+03	0.1319E+00	0.9308E+01	0.5000E-01
0.1945E+03	0.1711E+00	0.1396E+02	0.7500E-01
0.2037E+03	0.2013E+00	0.1862E+02	0.1000E+00
0.2216E+03	0.3631E+00	0.3233E+02	0.2500E+00
0.2193E+03	0.6142E+00	0.4503E+02	0.5000E+00
0.2236E+03	0.7424E+00	0.4933E+02	0.6250E+00
0.2259E+03	0.9003E+00	0.5162E+02	0.7812E+00
0.2290E+03	0.1321E+01	0.5486E+02	0.1200E+01

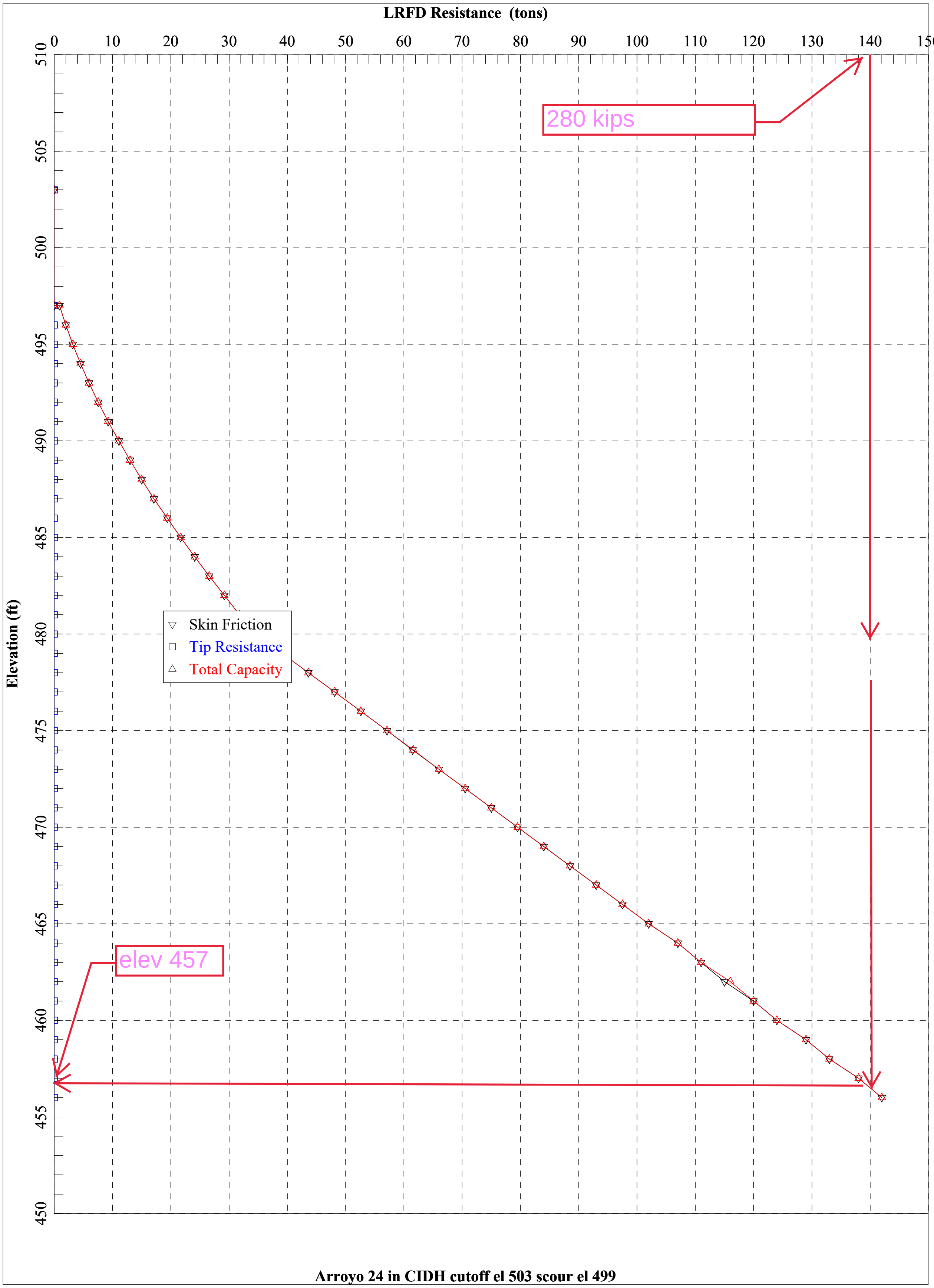
RESULT FROM UPPER-BOUND LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.9269E-01	0.5118E-04	0.2781E-02	0.1000E-04
0.4634E+00	0.2559E-03	0.1390E-01	0.5000E-04
0.9269E+00	0.5118E-03	0.2781E-01	0.1000E-03
0.4751E+02	0.2603E-01	0.1390E+01	0.5000E-02
0.7082E+02	0.3899E-01	0.2086E+01	0.7500E-02
0.9308E+02	0.5172E-01	0.2781E+01	0.1000E-01
0.1723E+03	0.1069E+00	0.6952E+01	0.2500E-01
0.2045E+03	0.1501E+00	0.1390E+02	0.5000E-01
0.2189E+03	0.1839E+00	0.2086E+02	0.7500E-01
0.2297E+03	0.2160E+00	0.2781E+02	0.1000E+00
0.2448E+03	0.3773E+00	0.4294E+02	0.2500E+00
0.2411E+03	0.6268E+00	0.5194E+02	0.5000E+00
0.2437E+03	0.7537E+00	0.5453E+02	0.6250E+00
0.2448E+03	0.9108E+00	0.5564E+02	0.7812E+00
0.2455E+03	0.1330E+01	0.5627E+02	0.1200E+01

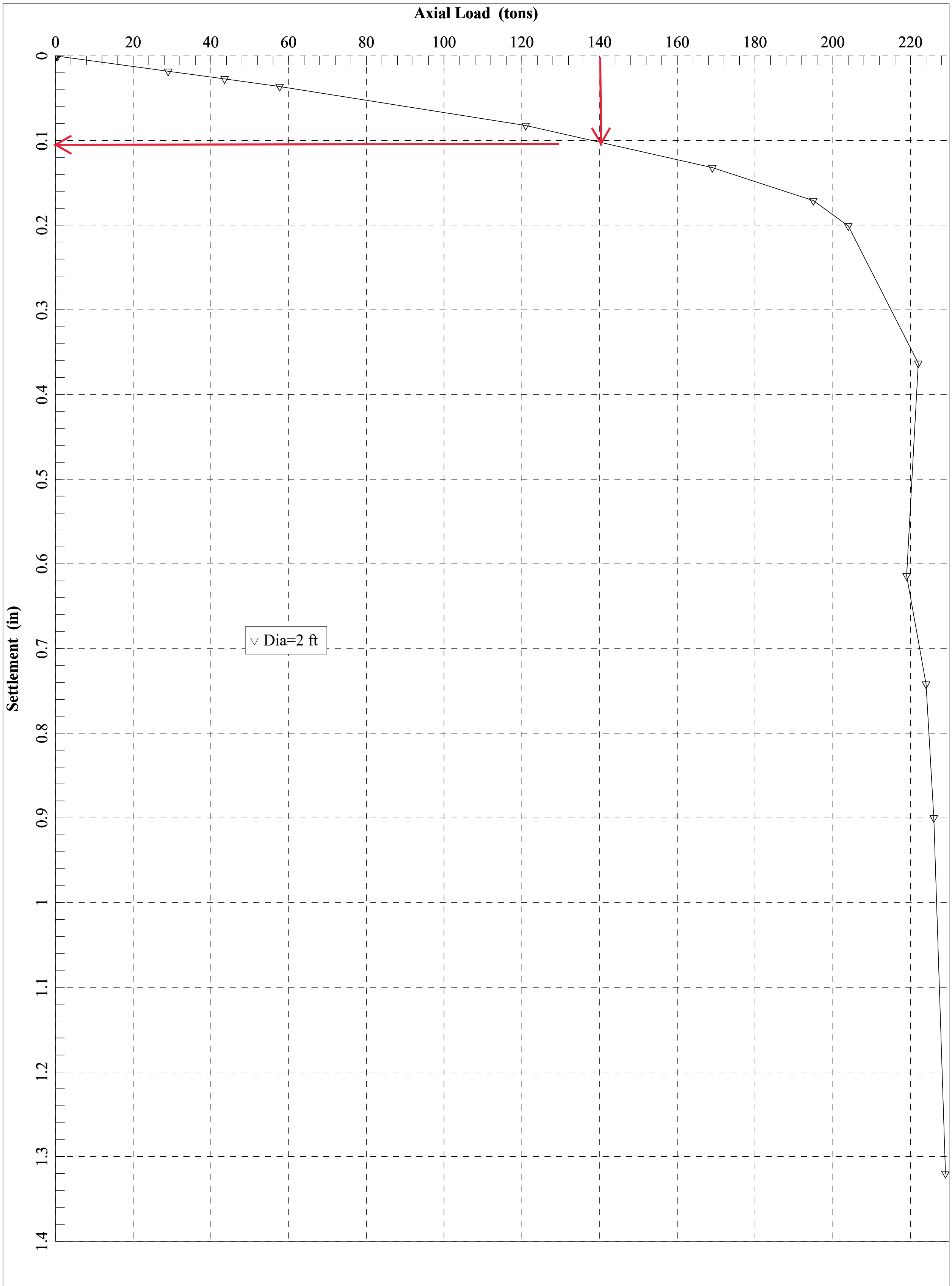
RESULT FROM LOWER-BOUND LINE

TOP LOAD TON	TOP MOVEMENT IN.	TIP LOAD TON	TIP MOVEMENT IN.
0.2909E-01	0.2357E-04	0.9426E-03	0.1000E-04
0.1455E+00	0.1178E-03	0.4713E-02	0.5000E-04
0.2909E+00	0.2357E-03	0.9426E-02	0.1000E-03
0.1469E+02	0.1184E-01	0.4713E+00	0.5000E-02
0.2204E+02	0.1776E-01	0.7070E+00	0.7500E-02
0.2939E+02	0.2368E-01	0.9426E+00	0.1000E-01
0.7005E+02	0.5800E-01	0.2356E+01	0.2500E-01

0.1209E+03	0.1079E+00	0.4713E+01	0.5000E-01
0.1533E+03	0.1492E+00	0.7070E+01	0.7500E-01
0.1720E+03	0.1835E+00	0.9426E+01	0.1000E+00
0.1984E+03	0.3489E+00	0.2173E+02	0.2500E+00
0.1975E+03	0.6016E+00	0.3813E+02	0.5000E+00
0.2035E+03	0.7310E+00	0.4414E+02	0.6250E+00
0.2068E+03	0.8897E+00	0.4753E+02	0.7812E+00
0.2122E+03	0.1313E+01	0.5316E+02	0.1200E+01



LRFD Resistance (tons)
Elevation (ft)
Skin Friction
Tip Resistance
Total Capacity
280 kips
elev 457
Arroyo 24 in CIDH cutoff el 503 scour el 499



Arroyo 24 in CIDH cutoff el 503 scour el 499

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Appendix IV.4 Pavement Analysis

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Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00

Project Cost: 0.00
Project Cost/Lane Mile: NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7**%
Design Life: **20** yrs
First Year Loads / Lane:
- Axles: **15,341**
- Trucks: **5,345**
- ESALs: **121,010**
- TI: **5.5**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks
No problems with minimum/maximum thickness checks;

Structural Adequacy Checks
Warning: Gravel Equivalent Provided above Layer 2 (AB): 0.73 is more than required: 0.59;
Warning: Gravel Equivalent Provided above Layer 3 (SG): 1.88 is more than required: 1.23;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.25	0.55	0.00	2.42	-0.02	0.80	0	
2	0.30	0.45	0.00	2.42	-0.01	0.75	0	
3	0.35	0.35	0.00	2.42	0.00	0.70	0	
4	0.40	0.35	0.00	2.42	0.12	0.75	0	
5	0.45	0.35	0.00	2.42	0.24	0.80	0	
6	0.50	0.35	0.00	2.42	0.36	0.85	0	
7	0.55	0.35	0.00	2.45	0.50	0.90	0	

CalFP-Web v3.DD001.3 **User:** David Kitzmann **Report Time Stamp:** Thursday, August 27, 2020, 4:40 PM

Project: Arroyo Road
Description: Arroyo Road Bridge Replacement
Trial: Conventional HMA
Description: HMA over AB

Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00

Project Cost: 0.00
Project Cost/Lane Mile: NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7%**
Design Life: **20** yrs
First Year Loads / Lane:
Axles: **15,341**
Trucks: **5,345**
ESALs: **121,010**
TI: **6.0**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks

No problems with minimum/maximum thickness checks;

Structural Adequacy Checks

Warning: Gravel Equivalent Provided above Layer 2 (AB): 0.69 is more than required: 0.62;

Warning: Gravel Equivalent Provided above Layer 3 (SG): 1.85 is more than required: 1.34;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.25	0.70	0.00	2.31	0.00	0.95	0	
2	0.30	0.60	0.00	2.31	0.01	0.90	0	
3	0.35	0.50	0.00	2.31	0.02	0.85	0	
4	0.40	0.40	0.00	2.31	0.02	0.80	0	
5	0.45	0.35	0.00	2.31	0.08	0.80	0	
6	0.50	0.35	0.00	2.31	0.20	0.85	0	
7	0.55	0.35	0.00	2.34	0.33	0.90	0	
8	0.60	0.35	0.00	2.41	0.49	0.95	0	

CalFP-Web v3.DD001.3 **User:** David Kitzmann **Report Time Stamp:** Thursday, August 27, 2020, 4:39 PM
Project: Arroyo Road
Description: Arroyo Road Bridge Replacement
Trial: Conventional HMA
Description: HMA over AB

Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00
							Project Cost:	0.00
							Project Cost/Lane Mile:	NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7**%
Design Life: **20** yrs
First Year Loads / Lane:
Axles: **15,341**
Trucks: **5,345**
ESALs: **121,010**
TI: **6.5**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks

No problems with minimum/maximum thickness checks;

Structural Adequacy Checks

Warning: Gravel Equivalent Provided above Layer 2 (AB): 0.67 is more than required: 0.66;

Warning: Gravel Equivalent Provided above Layer 3 (SG): 1.82 is more than required: 1.46;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.30	0.70	0.00	2.22	-0.02	1.00	0	
2	0.35	0.60	0.00	2.22	-0.02	0.95	0	
3	0.40	0.50	0.00	2.22	-0.02	0.90	0	
4	0.45	0.40	0.00	2.22	-0.02	0.85	0	
5	0.50	0.35	0.00	2.22	0.04	0.85	0	
6	0.55	0.35	0.00	2.25	0.17	0.90	0	
7	0.60	0.35	0.00	2.32	0.32	0.95	0	
8	0.65	0.35	0.00	2.38	0.47	1.00	0	

Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00

Project Cost: 0.00
Project Cost/Lane Mile: NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7%**
Design Life: **20** yrs
First Year Loads / Lane:
- Axles: **15,341**
- Trucks: **5,345**
- ESALs: **121,010**
- TI: **7.0**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks

No problems with minimum/maximum thickness checks;

Structural Adequacy Checks

Error: Gravel Equivalent Provided above Layer 2 (AB): 0.64 is less than required: 0.69;

Warning: Gravel Equivalent Provided above Layer 3 (SG): 1.80 is more than required: 1.57;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.30	0.85	0.00	2.14	0.01	1.15	0	
2	0.35	0.75	0.00	2.14	0.01	1.10	0	
3	0.40	0.65	0.00	2.14	0.00	1.05	0	
4	0.45	0.55	0.00	2.14	0.00	1.00	0	
5	0.50	0.45	0.00	2.14	0.00	0.95	0	
6	0.55	0.35	0.00	2.17	0.01	0.90	0	
7	0.60	0.35	0.00	2.23	0.16	0.95	0	
8	0.65	0.35	0.00	2.29	0.31	1.00	0	
9	0.70	0.35	0.00	2.35	0.46	1.05	0	

Project: Arroyo Road
Description: Arroyo Road Bridge Replacement
Trial: Conventional HMA
Description: HMA over AB

Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00

Project Cost: 0.00
Project Cost/Lane Mile: NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7%**
Design Life: **20** yrs
First Year Loads / Lane:
- Axles: **15,341**
- Trucks: **5,345**
- ESALs: **121,010**
- TI: **7.5**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks

No problems with minimum/maximum thickness checks;

Structural Adequacy Checks

Error: Gravel Equivalent Provided above Layer 2 (AB): 0.62 is less than required: 0.73;

Warning: Gravel Equivalent Provided above Layer 3 (SG): 1.78 is more than required: 1.68;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.35	0.85	0.00	2.07	-0.02	1.20	0	
2	0.40	0.75	0.00	2.07	-0.03	1.15	0	
3	0.45	0.70	0.00	2.07	0.02	1.15	0	
4	0.50	0.60	0.00	2.07	0.02	1.10	0	
5	0.55	0.50	0.00	2.09	0.02	1.05	0	
6	0.60	0.35	0.00	2.16	0.00	0.95	0	
7	0.65	0.35	0.00	2.21	0.14	1.00	0	
8	0.70	0.35	0.00	2.27	0.29	1.05	0	
9	0.75	0.35	0.00	2.32	0.45	1.10	0	

CalFP-Web v3.DD001.3 **User:** David Kitzmann **Report Time Stamp:** Thursday, August 27, 2020, 4:37 PM
Project: Arroyo Road
Description: Arroyo Road Bridge Replacement
Trial: Conventional HMA
Description: HMA over AB

Problem Description (User Input)

Project Location

District 4, Alameda, Route 84U, North, Start PM: 27.746, End PM: 28.714

Pavement Structure

Layer	Material	Thick (ft)	Modulus (ksi)	Poisson	R	GF	Cost ($/ft3)	Cost ($)
1	2020 Standard Old HMA for non-PRS Projects	0.30	1321.0	0.35	N/A	0.00	7.48	0.00
2	2020 Standard AB-Class 2 for non-PRS Projects	1.05	45.0	0.35	78	1.10	0.00	0.00
3	2020 Standard SM	0.00	17.7	0.35	30	0.00	0.00	0.00

Project Cost: 0.00
Project Cost/Lane Mile: NaN

Traffic Segment Counts

Design Lane Traffic Loads

Load Distribution (WIM Station): **Group1a**
Growth Rate (From First Year): **5.7%**
Design Life: **20** yrs
First Year Loads / Lane:
Axles: **15,341**
Trucks: **5,345**
ESALs: **121,010**
TI: **8.0**

Climate

Zone: **Inland Valley**

Results of the Caltrans Empirical Design Check Applied to the Current Structure

Minimum and Maximum Thickness Checks

No problems with minimum/maximum thickness checks;

Structural Adequacy Checks

Error: Gravel Equivalent Provided above Layer 2 (AB): 0.60 is less than required: 0.76;

Error: Gravel Equivalent Provided above Layer 3 (SG): 1.76 is less than required: 1.79;

CalFP Design Alternatives

Design	HMA	AB	SG	AC GF	Res GE	TtlThick	Cost/mi	MsgsText
1	0.40	0.90	0.00	2.00	0.00	1.30	0	
2	0.45	0.80	0.00	2.00	-0.01	1.25	0	
3	0.50	0.70	0.00	2.00	-0.02	1.20	0	
4	0.55	0.60	0.00	2.03	-0.02	1.15	0	
5	0.60	0.50	0.00	2.09	0.01	1.10	0	
6	0.65	0.35	0.00	2.14	-0.01	1.00	0	
7	0.70	0.35	0.00	2.20	0.13	1.05	0	
8	0.75	0.35	0.00	2.25	0.28	1.10	0	
9	0.80	0.35	0.00	2.30	0.43	1.15	0	