APPENDIX J

GEOTECHNICAL REPORT

Geotechnical Investigation

San Bernardino Class 1 Bike Trail Project

El Dorado County, California

July 26, 2019

Prepared for
NCE

Prepared By
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Mr. David Rios, CPESC, CPSWQ
NCE
P.O. Box 1760
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July 26, 2019
Project No.: 5012-02-1

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RE: Geotechnical Investigation
San Bernardino Class 1 Bike Trail Project
El Dorado County, California

Dear Mr. Rios:

Corestone Engineering, Inc. is pleased to present the results of our geotechnical investigation for the above-referenced project. Our investigation consisted of research, field exploration, laboratory testing, and engineering analysis to allow formulation of geotechnical conclusions and recommendations for design and construction of the proposed shared-use path project.

The San Bernardino Class 1 Bike Trail Project will construct approximately 1,200 linear feet of new asphalt concrete paved shared-use path between West San Bernardino Avenue and East San Bernardino Avenue. The path will cross the Upper Truckee River on a new bridge.

Site subsurface soils are almost exclusively granular sandy soils which will provide excellent support for the proposed embankment to host the path as well as the proposed bridge. Relatively thin layers of potentially liquefiable, loose sand soils exist in the areas of the proposed bridge footings. We estimate about 1 inch of liquefaction-induced seismic settlement associated with these layers to the shallow, spread footings of the bridge. Geotechnical design and recommendations for bridge foundations included in this report should be finalized once final details on the proposed bridge become available.

We appreciate having the opportunity to work with you on this project. If you have any questions regarding the content of the attached report, please do not hesitate to contact us.

Sincerely,

Corestone Engineering, Inc.

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1.0 Introduction

Presented herein are the results of Corestone Engineering, Inc.'s (CEI's) geotechnical investigation, laboratory testing, and associated geotechnical design recommendations for the San Bernardino Class 1 Bike Trail Project to be located near the Meyers community area in El Dorado County, California. These recommendations are based on surface and subsurface conditions encountered in our explorations and on details of the proposed project as described in this report. The objectives of this study were to:

1. Determine general soil and groundwater conditions pertaining to design and construction of the proposed new shared-use path, including a bridge crossing at the Upper Truckee River.

2. Provide recommendations for design and construction of the project as related to these geotechnical conditions.

Our investigation included field exploration, laboratory testing, and engineering analysis to determine the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations.

The services described above were conducted in accordance with the Master Subconsultant Agreement No. SC211-17 between NCE and CEI dated March 1, 2017, and NCE's work authorization for the project dated November 14, 2018.

2.0 Project Description

2.1 Project Location and Existing Facilities

The proposed project alignment spans between the current terminations of West San Bernardino Avenue and East San Bernardino Avenue and is approximately 1,200 feet long. The site is entirely contained in Section 30, Township 12 North, Range 18 East, Mount Diablo Meridian in El Dorado County, California. The site is located within forested land, with residential properties on the west end and along West San Bernardino Avenue and Tahoe Paradise Park on the east end at the current terminus of East San Bernardino Avenue. The approximate latitude and longitude of the project site at the western end is 38.85621 and -120.02996, respectively, from Google Earth™. Access to the site is obtained by West San Bernardino Avenue or by East San Bernardino Avenue and then through the park.

2.1 Proposed Project Details

Only conceptual project details were available at the time of this report. The San Bernardino Class 1 Bike Trail Project will construct approximately 1,200 linear feet of new shared-use path for utilization by both bicyclists and pedestrians. The path will cross the Upper Truckee River, located west of Tahoe Paradise Park, on a new bridge. The shared-use path will include asphalt concrete surfacing. The path alignment is expected to be slightly raised throughout, and the bridge approach embankment is expected to include as much as 10 feet of embankment fill. The width of the path is expected to be 10 feet and may increase slightly at the bridge. The path will essentially connect West San Bernardino Avenue and East San Bernardino Avenue, providing a continuous access for bicyclists and pedestrians.

Final bridge alignment, bridge length, bridge type, structural loads, and substructure details were not available at the time of this report. At this time, 2 bridge alignment alternates are being considered; the first alignment alternate will provide a straight connection extension of the existing trail alignment on the west side of the Upper Truckee River, and the second alternate will place the bridge slightly to the north/northwest approximately 50 feet from the first alignment. It is our understanding El Dorado County will begin the design process later this year. Based on our discussion with El Dorado County, the bridge is expected to be 200 feet in length. We assume the bridge will be a 3-span structure supported on 2 end abutments and 2 intermediate piers. The middle span is expected to be longer than the 2 other spans and will cross over the normal water limits of the Upper Truckee River, with piers located on either side of the river.

The proposed bridge structure will be designed and constructed per the California Department of Transportation (Caltrans) standards utilizing the Load Resistance Factor Design (LRFD) method. In particular, the currently applicable American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications*, 8th Edition (AASHTO, 2017) will be used in the design.

3.0 Site Conditions and Regional Settings

3.1 Site Conditions

The project site runs approximately west-southwest to east-northeast from the eastern end of West San Bernardino Avenue to the western end of East San Bernardino Avenue within the Tahoe Paradise Park. The site crosses the Upper Truckee River in a pine forest. Currently, a footpath is present east of the river and an unimproved road is present west of the river (utility easement). The topography across the eastern and central portions of the site has low vertical relief and slopes very gently towards the Upper Truckee River. The western edge of the site has a moderate slope approximately 12 feet high and includes embankment fill up to approximately 8 feet thick.

Underground utilities, including sanitary sewer, water, and communications, are present within the shared-use path alignment. Sewer is present within the western half of the site, while communications and water are present throughout the site. Communications and waterline cross the Upper Truckee River and are protected from upstream erosion by a sheet-pile wall located just north of the river crossing.

The overall site is located within a pine forest with mature pine trees.

3.2 Regional Geology and Seismicity

The project is located in the Tahoe Basin of the Sierra Nevada mountains. The Tahoe Basin is within the Sierra Nevada Batholith consisting of mainly massive, Cretaceous age granitic rock subsequently overlain by Tertiary age volcanic and volcaniclastic rock. Within late Tertiary and Quaternary time, Basin and Range style extensional regional faulting has extended into the Sierra Nevada, and the Tahoe Basin is a fault-bounded basin at the western edge of Basin and Range faulting. The current landscape has been shaped by an extensive Pleistocene age glacial history and continues to be shaped by fluvial and lacustrine processes and active faulting. Because of its geological settings, the Tahoe Basin has a high potential for strong seismic shaking.

4.0 Exploration

4.1 Drilling

The San Bernardino Class 1 Bike Trail Project site was explored on May 21 and 22, 2019, by drilling 8 test borings. The locations of the borings are shown on Plate 1 (Location of Borings Map). A well/drilling permit was obtained from El Dorado County Environmental Management Department to complete the exploration borings. The borings were drilled using 4-inch-outside-diameter (O.D.), solid-stem augers and a track-mounted CME 55 soils sampling drill rig. Where groundwater prevented solid-stem auger drilling or undisturbed blow counts were necessary, HQ coring or mud-rotary drilling techniques were used. The maximum depth of exploration was 41.5 feet below the existing ground surface.

The native soils were sampled in-place every 1.5 to 2.5 feet by use of a standard, 2-inch-O.D., split-spoon sampler driven by a 140-pound safety drive hammer with a 30-inch stroke operated with a rope and cathead. The number of blows to drive the sampler the final 12 inches of an 18-inch penetration (Standard Penetration Test [SPT] - American Society for Testing and Materials [ASTM] D 1586) into undisturbed soil is an indication of the density and consistency of the material.

A 3-½-inch-O.D., split-spoon sampler (ASTM D 3550) was also used to sample soils containing gravel or where approximate in-place densities of subsurface materials were required. Sampling methods used were similar to the SPT but also included the use of 2-½-inch-diameter, 6-inch-long, brass sampling tubes placed inside the split-spoon sampler. Because of the larger diameter of the sampler, blow counts are typically higher than those obtained with the SPT and should not be directly equated to SPT blow counts. The logs indicate the type of sampler used for each sample.

Groundwater levels were measured where encountered in the borings at the time of exploration.

4.2 Material Classification

A geologist examined and identified all soils in the field in accordance with ASTM D 2488 and the Caltrans (2010) Logging Manual. During drilling exploration, representative bulk samples were placed in sealed plastic bags and returned to Reno, Nevada, for testing. Additional soil classification was subsequently performed in accordance with ASTM 2487 (Unified Soil Classification System [USCS]) upon completion of laboratory testing, as described in the **Laboratory Testing** section. A soil classification chart is included in Appendix A-1 (USCS Soil Classification Chart). Logs of the test borings are presented as Appendix A-2 (Boring Logs).

5.0 Laboratory Testing

All soils testing performed was conducted in general accordance with the standards and methodologies described in Volume 4.08 of the ASTM Standards and the California Test Methods (CTM), as appropriate. Laboratory testing was performed by Black Eagle Consulting, Inc. of Reno, Nevada.

5.1 Index Tests

Samples of each significant soil type were analyzed to determine their in-situ moisture content (ASTM D 2216), grain size distribution (ASTM D 422), and plasticity index (ASTM D 4318). The results of these tests are shown on Appendix B-1 (Index Test Results). Test results were used to classify the soils according to ASTM D 2487 and to verify field logs, which were then updated as appropriate. Classification in this manner provides an indication of the soil's mechanical properties and can be correlated with standard penetration testing and published charts (Bowles, 1996; Naval Facilities Engineering Command [NAVFAC], 1986a and b) to evaluate bearing capacity, lateral earth pressures, and settlement potential.

5.2 Direct Shear Tests

Two direct shear tests (ASTM D 3080) were performed on representative samples of subsurface soils in the proposed bridge area. The tests were run on remolded, inundated samples under various normal loads in order to develop a Mohr's strength envelope. For remolded samples, the samples were screened to remove particles larger than the number 4 sieve prior to testing. Results of these tests are shown on Appendix B-2 (Direct Shear Test Results) and were used in calculation of bearing capacities, friction factors, and lateral earth pressures.

5.3 R-Value Tests

Two resistance value (R-value) tests (CTM 301) were performed on representative samples of subgrade soil that will be present along the pathway. Resistance value testing is a measure of subgrade strength and expansion potential and is used in design of flexible pavements. Results of the R-value tests are shown on Appendix B-3 (R-Value Test Results).

5.4 Chemical Tests

Chemical testing was performed on representative samples of site foundation soils to evaluate the site materials' potential to corrode steel and Portland cement concrete in contact with the ground. The samples were tested for pH, resistivity, redox potential, soluble sulfates, and sulfides. The results of the chemical tests are shown on Appendix B-4 (Chemical Test Results). Chemical testing was performed by Silver State Analytical Laboratories of Reno, Nevada.

6.0 Site Geology and Subsurface Conditions

6.1 Site Geology

Mapping by the California Geological Society (CGS) indicates the site is located within Pleistocene age *Tahoe glacial deposits - Till* (Saucedo, 2005). These materials are described by the CGS as *unsorted to very poorly sorted, boulder to clayey gravel; surface granitic boulders slightly to moderately weathered. Associated with undissected to moderately dissected moraines. Locally may include outwash deposits.* Due to the site's proximity to the Upper Truckee River, the majority of soils encountered include well-sorted sands and silty sand fluvial deposits, with remnant glacial deposits encountered at the eastern and western ends of the project.

6.2 Subsurface Soil Conditions

The soils profile throughout the site typically consists of surficial silty to poorly graded sand with some gravel through 5 feet depth below existing ground surface and through a slightly deeper horizon (12.5 feet) near the Upper Truckee River. Beneath the gravelly soils are silt or very fine silty sand soils from about 5 to 10 feet beneath the ground surface. The underlying soils consist of fine to medium silty sand through the maximum depth of exploration, 41.5 feet beneath the existing ground surface.

The surficial gravelly soils are brown, moist to wet, loose to medium dense, and contain about 5 to 20 percent non-plastic fines and 0 to 35 percent subrounded to rounded gravel. The intermediate depth fine silty sand to silt soils are gray to light gray, wet, stiff (loose to medium dense), and contain 30 to 90 percent non-plastic fines and 10 to 70 percent very fine to fine sand. The underlying soils are relatively uniform to 41.5 feet depth and are described as light gray, moist, loose to dense, and as containing approximately 10 to 30 percent non-plastic fines, 70 to 90 percent fine to coarse sand, and trace amounts of fine gravel.

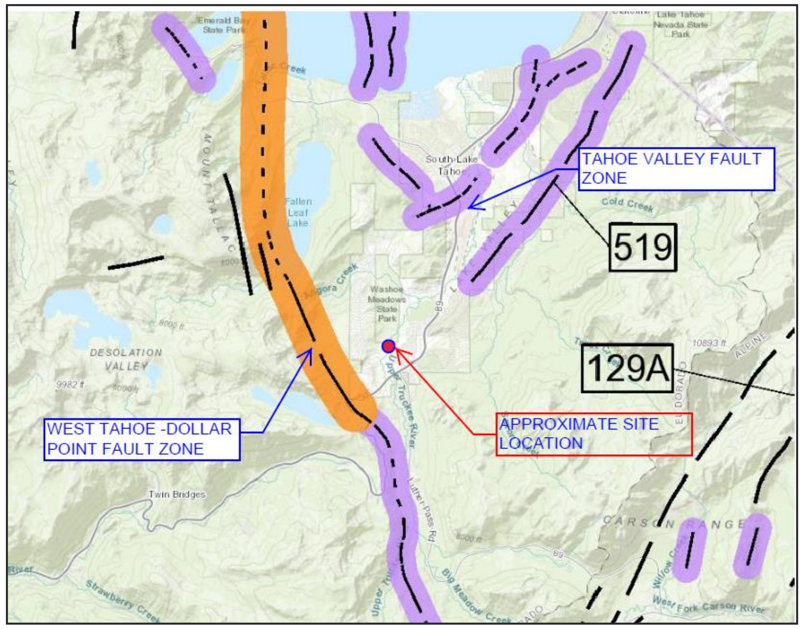
6.3 Groundwater

Groundwater was encountered in each boring advanced at the time of exploration at variable depths of approximately 1.5 to 7 feet below the existing ground surface. The depth to groundwater generally becomes shallower towards the Upper Truckee River, and near the river the groundwater matched the river water level. These groundwater depths correspond to approximate elevations of between 6,292 feet above mean sea level (msl) and 6,303 feet above msl. Fluctuations in the groundwater table will occur due to rainfall, temperature, seasonal runoff, Upper Truckee River water level, adjacent irrigation practices, and other factors. Groundwater near the Upper Truckee River will generally be controlled by the river water level.

7.0 Geologic Hazards

7.1 Seismicity and Faults

The Lake Tahoe basin lies within an area with a high potential for earthquake shaking. It is generally accepted that a maximum credible earthquake in this area would be in the range of magnitude 7 to 7.5 along the Genoa fault system of the eastern Sierra Nevada. The most active segment of this fault system in the Tahoe area is located at the base of the eastern side of the mountains, about 7 miles east of the project.



Fault Activity Map

No known faults are mapped through or in the immediate vicinity of the proposed shared-use path alignment (CGS, 2019a). The mapped faults in the general area of the project site are shown on the figure to the right. The nearest mapped fault is a Holocene age (less than 11,700 years old) fault segment associated with the West Tahoe – Dollar Point fault zone and is mapped about 1 mile west of the project site. This fault zone is also identified as an Alquist-Prioloa Earthquake Fault Zone (EFZ) by the CGS (2019b). Because the site does not lie within an AFZ and no unmapped faults were suggested within the site during our investigation, no additional fault investigation or mitigation is considered necessary for the proposed project.

7.2 Ground Rupture

There are no known ground rupture locations within or in the general area of the project site. The absence of ground faults passing through or in the immediate vicinity of the site suggests the potential for ground rupture within the project site is negligible.

7.3 Ground Motion

The Caltrans *ARS Online* web-based tool was utilized in determining the design response spectrum for the site considering both deterministic and probabilistic acceleration spectra (Caltrans, 2019a). Based on the analysis, the peak ground acceleration (PGA) for the site is 0.42 g. Detailed discussion on the parameters utilized in determining the design response spectrum for the site are included later under the Seismic Design Criteria section.

7.4 Liquefaction

Based on our site exploration, the site is underlain by a shallow groundwater table and submerged, isolated, loose to medium dense sand layers. Therefore, a detailed liquefaction analysis is warranted to determine the liquefaction hazard at the site and to quantify liquefaction-induced settlement at the bridge footing grade.

Liquefaction analysis of the site area to host the proposed bridge was performed using subsurface information obtained from boring B-02 which was advanced using mud rotary drilling techniques. The analysis was performed using the methods and procedures recommended by ASTM D 6066 and the SPT-based liquefaction analysis method recommended by Idriss and Boulanger (2008); these procedures are generally consistent with liquefaction evaluation guidelines of the Caltrans Geotechnical Manual (Caltrans, 2016). The peak ground acceleration used in the liquefaction analysis is 0.42 g, as noted earlier. This value is equal to the design acceleration value at zero period from the design spectrum for the site. The earthquake deaggregation analysis (United States Geological Survey, 2008) resulted in a maximum earthquake magnitude of 6.48 for the site, which is the magnitude that was used in the liquefaction analysis. The groundwater level during earthquake loading was assumed to be at 2.5 feet below existing ground surface. Liquefaction analysis calculations are shown on Appendix C (Analysis Calculations).

Based on our liquefaction analysis, there are 2 approximately 2.5-foot-thick, potentially liquefiable sand soil layers present at depths of 5 and 15 feet below existing ground surface at boring B-02 which was advanced on the east side of the Upper Truckee River. These potentially liquefiable layers exhibit factor of safety values in the range of 0.5 to 0.7 with respect to liquefaction for the design earthquake. These layers have "clean sand" penetration resistance values of approximately 12 to 20 blows per foot. Boring B-03 on the west side of the river was advanced utilizing solid-stem auger drilling and coring techniques and, therefore, the data from this boring were not analyzed (considering some sample disturbance). However, boring B-03 also shows a loose sand layer at a shallow depth of 2.5 feet below the existing ground surface and confirms that the entire bridge site exhibits shallow layers of potentially liquefiable soils.

The liquefaction analysis further shows that liquefaction of the above-discussed 2 layers with the occurrence of a design earthquake on a nearby fault could cause a total liquefaction-induced settlement of approximately 2 inches at the horizons of these layers (volumetric change in the liquefiable soil layers). However, the surface manifestation and liquefaction-induced damage at the ground level will depend on the peak ground acceleration, the thicknesses of liquefiable soil layers, thicknesses of non-liquefiable soil layers in between liquefiable soil layers, and the thickness of non-liquefiable deposits above the top of the first liquefiable soil layer. Based on the criteria developed by Ishihara (1985), and using the locations and thicknesses of liquefiable soil layers within the site, we expect minimal manifestation of liquefaction-induced settlement at the bridge footing level from the relatively thin potentially liquefiable layer at 15 feet depth below existing ground surface. However, the potentially liquefiable layer at the shallower depth is expected to result in up to 1 inch of liquefaction-induced seismic settlement to the bridge foundation, particularly pier footings that are to be founded below the existing ground surface.

7.5 Flood Plains and Scour Evaluation

The Federal Emergency Management Agency (FEMA) has identified the site as lying in Zone AE, or within the limits of a 100-year flood plain with a base flood elevation of 6,301 to 6,304 feet above msl within the vicinity of the Upper Truckee River crossing (FEMA, 2008).

Information with respect to scour associated with the Upper Truckee River was not available at the time of this report. It is our understanding hydraulic studies will be performed to determine the scour depth to establish bridge foundation depths near the Upper Truckee River. The shallow foundations to support bridge piers near the Upper Truckee River will be founded below the scour depth.

7.6 Other Geologic Hazards

A moderate to high potential for dust generation is present if the embankment construction is performed in dry weather.

The site is relatively flat; as such, no landslides should occur.

No other geologic hazards were identified.

8.0 Conclusions and Recommendations

The site is geotechnically suitable to host the San Bernardino Class 1 Bike Trail Project. The following summarizes our conclusions:

- The site is overlain by granular sand soils which will provide adequate support for the proposed shared-use path, including the bridge across the Upper Truckee River.
- The site is located in an area with high potential for strong earthquake shaking. The proposed bridge site exhibits relatively thin layers of loose sand soils which are potentially liquefiable for the design earthquake event. We estimate approximately 1 inch of liquefaction-induced seismic settlement to bridge footings and approach embankment due to liquefaction of the sandy soil layer that exists through about 7.5 feet below the existing ground surface.
- Shallow, spread footings are feasible for bridge support and will likely be the most economical foundation type. Depending on the footing depth, at least a portion of the loose sand soils at relatively shallow depths will be densified and this will further reduce the expected seismic settlement.
- Groundwater throughout the site is shallow and was encountered at depths of about 1.5 to 7 feet below the existing ground surface. The construction of bridge pier footings will likely require dewatering. Submerged sand soils will be saturated and impossible to compact; stabilization measures should be anticipated. Construction should consider seasonal groundwater variations.

Final bridge alignment, bridge length, bridge type, structural loads, and substructure details were not available at the time of this report. The geotechnical design and recommendations provided for the bridge foundations and other associated structural elements shall be considered preliminary. Once design information becomes available, CEI must be provided the opportunity to review the information and provide any needed update to the recommendations.

Any evaluation of the site for the presence of surface or subsurface hazardous substances is beyond the scope of this investigation. When suspected hazardous substances are encountered during routine geotechnical investigations, they are noted in the exploration logs and immediately reported to the client. No such substances were revealed during our exploration.

8.1 Seismic Design Criteria

As noted earlier under **Ground Motion** (Section 7.3), the Caltrans *ARS Online* web-based tool was utilized in determining the design response spectrum for the site (Caltrans, 2019a). The design response spectrum is developed considering both deterministic and probabilistic acceleration spectra. Based on our boring exploration for the bridge, the site soils are generally medium dense sand soils with SPT blow counts greater than 15. Based on this information and the site geology, a Site class D soil profile is appropriate to develop seismic design criteria. The Site Class D soil profile is for stiff soils with a shear velocity between 600 and 1,200 feet per second (approximately 180 meters per second [m/s] to 360 m/s) , or with an N (SPT) value between 15 and 50, or an undrained shear strength between 1,000 and 2,000 pounds per square foot (psf). Table 1 (Seismic Design Criteria Site Parameters) provides the site and soil parameters utilized in developing seismic criteria using the Caltrans *ARS Online* tool, and the developed design response spectrum is included as Plate 2 (Seismic Design Data).

TABLE 1 – SEISMIC DESIGN CRITERIA SITE PARAMETERS

Parameters		Value
Site Location	Latitude	38.85728
	Longitude	-120.02702
Site Class		D
Shear Wave Velocity		270 m/s[1]

[1] Default value for Site Class D soil profile in the Caltrans *ARS Online* tool is selected and is appropriate based on the SPT blow counts.

The seismic design criteria for the site utilizing the above parameters are provided in Table 2 (Seismic Design Criteria). It is noted that the Caltrans Seismic Design Criteria (SDC) manual also recommends the consideration of statewide minimum spectrum defined as the medium spectrum generated by a magnitude 6.5 earthquake on a strike-slip fault located 12 kilometers from the bridge site (Caltrans, 2019b). The proposed shared-use path bridge site is located closer than 12 kilometers to a fault with larger than the statewide minimums provided by the SDC.

TABLE 2 - SEISMIC DESIGN CRITERIA

Parameters	Design Acceleration (g)
PGA	0.419
Design Spectral Response at 0.2 Second	0.953
Design Spectral Response at 1.0 Second	0.660

8.2 Foundation Design

8.2.1 Foundation Type Selection

At this stage, it is our opinion the most economical way to support the proposed bridge is via shallow, spread foundations bearing on properly prepared native soils or densified embankment fill. Depending on the structural loads, bridge alignment and other final design conditions, deep foundations such as driven piles may also be considered to support the bridge. Any retaining walls to support the bridge approach embankment may also be founded on conventional shallow foundations. As discussed earlier, a potential for soil liquefaction exists at the site. However, with proper design, shallow foundations will perform adequately with tolerable seismic settlement to improvements.

8.2.2 Shallow Foundations Design

The design of shallow foundations was performed using the methods provided in Section 10.6 of the AASHTO *LRFD Bridge Design Specifications* 8th Edition (AASHTO, 2017). The theoretical bearing resistance was computed per Section 10.6.3 of AASHTO for footings bearing on sand utilizing the SPT method, and a resistance factor of 0.45 was applied for Strength Limit State design. Bearing capacity factors for footings founded near a slope were utilized for bridge abutment footings; a 2H:1V (horizontal to vertical) embankment fill slope and a minimum setback of 5 feet from the slope face for footing edges were assumed in the analyses. Based on the laboratory direct shear test results, native sand soils were assigned a conservative angle of internal friction of 36 degrees. Embankment fill materials were also assigned an angle of internal friction of 36 degrees. The site soils are cohesionless granular soils, and the settlement analysis was performed using the Hough method. Cohesive soils subject to long-term consolidation settlement do not exist at the site. Table 3 (Bearing Resistance for Spread Footings) provides geotechnical recommendations for spread foundations bearing on properly prepared native sand soils or densified embankment fill. Analysis calculations for spread footings are included as Appendix C.

TABLE 3 – BEARING RESISTANCE FOR SPREAD FOOTINGS					
Design Location and Conditions	Footing Width (feet)[1]	Minimum Embedment Depth (feet)	Factored Bearing Resistance (ksf*)		Service Limit State Bearing Resistance for 1 Inch Permissible Settlement (ksf*)
			Extreme Event Limit State	Strength or Construction Limit State	
Pier Footings Bearing on Native Soils[2]	5.0	3.0	8.0	3.6	4.7
	10.0	3.0	13.9	6.3	2.6
Abutment Footings above 2H:1V Embankment Fill Slope	5.0	3.0	3.9	1.7	11.7
	10.0	3.0	7.4	3.3	4.1

* ksf – kips per square foot.
[1] Analyses consider square and rectangular foundations with maximum footing length to width ratio of 2. Values may be interpolated for other footing widths.
[2] Values may also be utilized for retaining wall foundations.

For spread footings designed per the Table 3 recommendations, total foundation settlement should be 1 inch or less for Service Limit State loads. Differential movement between footings with similar loads, dimensions, and base elevations should not exceed two-thirds of the total settlement. The majority of the anticipated movement will occur during the construction period as loads are applied. As discussed earlier under Section 7.4 (Liquefaction), liquefaction-induced seismic settlement of approximately 1 inch is anticipated.

We assume cast-in-place spread footings will be utilized. Factored sliding resistance factors of 0.72 and 0.58 are appropriate for cast-in-place spread footings for Extreme Event Limit State and Strength Limit State design conditions, respectively. Resistance factors of 1.0 and 0.8 are considered for sliding resistance for Extreme Event Limit State and Strength Limit State, respectively.

A passive lateral earth pressure value (equivalent fluid pressure [EFP]) of 480 pounds per cubic foot (pcf) is appropriate for design of footings to calculate the passive earth pressure component of sliding resistance against lateral loads. This value assumes footings are backfilled with densified structural fill that meets the structure backfill specifications of Caltrans *Standard Specifications* (Caltrans, 2018). Passive earth pressure shall be neglected within 2 feet from the adjacent lowest grade. A resistance factor of 0.50 shall be applied to the passive earth pressure value for Strength Limit State design.

8.3 Lateral Earth Pressures

It is our understanding cast-in-place retaining walls (Caltrans Type 1 or Type 5) or segmental block walls (Keystone or other proprietary manufacturer) as tall as 10 feet will be utilized at the approaches to the bridge. Table 4

(Lateral Earth Pressure Recommendations) provides EFP values for design of retaining walls and also abutment back walls. Table 4 values are for fully drained retaining walls with vertical back faces, horizontal backfill, and no surcharge loads next to the top of the wall. Lateral earth pressure values due to surcharge loads are discussed later. These parameters also assume backfill material against abutments and retaining walls will meet Caltrans *Standard Specifications* of structure backfill (Caltrans, 2018).

TABLE 4 – LATERAL EARTH PRESSURE RECOMMENDATIONS

Parameters		Values
At Rest EFP	Static	52 pcf
	Seismic[1]	81 pcf
Active EFP[2]	Static	30 pcf
	Seismic[1]	47 pcf
Passive EFP[3]		480 pcf

[1] Total value includes static and additional seismic EFP.
[2] Active EFP shall only be used for walls that can deflect or move sufficiently to mobilize active conditions. Wall deflection/movement of at least 0.002 times the height of the active section of the wall is required to fully mobilize active pressure conditions.
[2] Full value of passive EFP shall only be used for walls that can deflect or move sufficiently to mobilize passive pressure conditions. Wall deflection/movement of at least 0.02 times the height of the passive section of wall is required to fully mobilize passive pressure conditions. In order to limit the deflection/movement, the value may be reduced by a factor of 1.5.

The EFP values provided in Table 4 were calculated per the AASHTO *LRFD Bridge Design Specifications* 8th Edition (AASHTO, 2017). A soil unit weight of 125 pcf was used to calculate EFP values from lateral earth pressure coefficients. The Mononabe-Okabe (M-O) equation (AASHTO, 2017) was used to calculate active lateral earth pressure coefficient for seismic loading. The horizontal seismic acceleration coefficient (Kh) of 0.21 was utilized in the analysis and is equal to half the value of the PGA per AASHTO design procedures. The at-rest active lateral earth pressure value for seismic loading was calculated by applying a similar ratio/level of increase in additional active lateral earth pressure values from static to seismic loading. The resultant of the EFP for static loading shall be applied at an H/3 height above the base of the wall where H is equal to the height of the wall. Per current AASHTO recommendations, routine retaining wall design for seismic loading may use the resultant of the EFP for seismic loading applied at an H/3 height above the base of the wall. Because the walls on this project will be associated with the proposed bridge, we recommend the resultant of the EFP for seismic loading be applied at a 0.4H height above the base of the wall.

Where necessary, surcharge loads shall be considered in the design of retaining walls. Lateral earth pressure values due to uniform surcharge loads shall be estimated utilizing active and at-rest lateral earth pressure coefficients of 0.24 and 0.42, respectively. The lateral earth pressure value for the selected design case (active or at-rest) will be calculated by multiplying the uniform surcharge load by the respective lateral earth pressure coefficient. In order to consider surcharge loads associated with maintenance vehicle loading, we recommend a uniform surcharge load equal to 240 psf be considered in the design of retaining walls; this value is based on the applied pressure from the weight of the 2-foot-high soil column with a unit weight of 120 pcf.

8.4 Structural Section Design for Class 1 Pathway

Based on our laboratory testing, the native sand and gravel soils are excellent subgrade materials exhibiting R-value in excess of 70. It is expected embankment fills will be placed to establish the design grades for the path in portions of the alignment, and the height of the embankment fills is expected to be as high as 10 feet at the approaches to the bridge. Therefore, subgrade of the pathway will consist of either densified native soils or embankment fills. A Traffic Index of 5.0 is appropriate for design of the proposed Class 1 pathway which will be subject to light loads from occasional maintenance vehicles. Based on the subgrade conditions and light load application, a minimum structural section consisting of 0.2 feet of asphalt concrete pavement underlain by 0.5 feet of Class 2 aggregate base is considered appropriate. The aggregate base shall be densified to at least 95 percent relative compaction, as determined per CTM 216.

8.5 Slope Stability and Erosion Control

Based on our investigation, new embankment fill side slopes constructed at 2H:1V or flatter will be globally stable at the site up to the expected maximum heights of 10 feet. Erosion protection via rip-rap or other methods should be considered for slopes steeper than 3H:1V.

8.6 Site/Subgrade Preparation

All vegetation and debris (including wood chips at the western end of the project alignment) shall be stripped and grubbed from structural areas and removed from the site. Trees and associated roots greater than ½ inch in diameter shall be removed, where necessary, to a minimum depth of 12 inches below finished grade. Large roots (greater than 6 inches in diameter) shall be removed to the maximum depth possible. Resulting excavations shall be backfilled with embankment fills compacted to 90 percent relative compaction per CTM 216.

Existing embankment fills are present at the western end of the project alignment. The thickness of these existing fills is as much as 8 feet, as encountered in our exploration, and includes relatively loose zones. We recommend existing fills be reworked through at least 2 feet depth to provide sufficient support for the proposed pathway. This reworking process will involve removal of existing fills through at least 12 inches depth below existing ground surface and then scarification of the exposed surface through an additional 12 inches depth, moisture conditioning, and compaction to at least 90 percent relative compaction per CTM 216. The removed embankment fills shall then be replaced and compacted per the requirements of embankment fill to establish design grades or to receive additional embankment fills.

All areas to receive embankment fills or structural loading shall be densified to at least 90 percent relative compaction per CTM 216.

If wet weather construction is anticipated or for excavations at and below the groundwater table, soils will be above optimum moisture and impossible to compact. In some situations, moisture conditioning may be possible by scarifying the top 12 inches of subgrade and allowing it to air-dry to near-optimum moisture prior to compaction. Where this procedure is ineffective or where construction schedules preclude delays, mechanical stabilization will be necessary. Mechanical stabilization can generally be achieved by removal of unstable soils through 12 inches depth, placing a geogrid layer, and then placement of Class 2 or Class 3 aggregate base (Caltrans, 2018). Aggregate base shall be placed in a single lift within 12 inches of over-excavation and densified to at least 90 percent relative compaction per CTM 216. Geogrid shall be Tensar® TX160 or an approved equivalent. In some cases where pumping of soils is significant, an intermediate, second geogrid layer may be necessary.

8.7 Grading and Embankment Construction

Site grading and earthwork shall follow Caltrans *Standard Specifications* (Caltrans, 2018).

The project will require minimum cuts, if any. Up to 10 feet of fills will be placed for embankment construction. Existing fills within the western limits of the project and excavated native sand soils will be suitable to reuse as embankment fills. Imported borrow will be required for the project. It is expected borrow will be imported from a nearby source. Imported borrow should meet the specifications for Class 3 Aggregate Subbase (Caltrans, 2018). Other granular, non-expansive materials approved by the geotechnical engineer may also be used as imported borrow. In no case shall expansive material (Expansion Index of 50 or greater and Sand Equivalent of 20 or less) be used as fills. Fill should be free of debris and organic material.

All embankment fills placed within 100 feet of the bridge shall be placed in maximum 8-inch-thick loose lifts each densified to at least 95 percent relative compaction per CTM 216. All other embankment fills shall be densified to a minimum 90 percent relative compaction per CTM 216.

8.8 Cuts and Excavation

No significant cuts are expected on the project. Temporary excavations and sloping will be necessary for footing construction and any utility installation. Temporary excavations with near-vertical sidewalls are not expected to be stable in the site materials and, as such, should be sloped or shored in accordance with Cal/OSHA requirements. All site soils are Type C and shall be sloped at 1.5H:1V or flatter in temporary excavations.

On-site materials excavated and compacted as embankment fills should experience quantity shrinkage of approximately 10 percent due to density increase.

8.9 Corrosion Evaluation

Corrosion testing was completed on a representative sample obtained from test borings advanced at the bridge site. Corrosion test results are summarized in Table 5 (Corrosion Test Results Summary), and detailed results are contained in Appendix B-4.

TABLE 5 – CORROSION TEST RESULTS SUMMARY					
Sample Identification	**Depth (feet)**	**pH**	**Minimum Resistivity (ohm-cm)**	**Chloride Content (ppm)**	**Sulfate Content (ppm)**
B-04 A	2.5	5.8	16,000	<150	<60

Based on the test results and Caltrans Corrosion Guidelines, the soils are non-corrosive to structural steel and concrete foundation elements in contact with soils. It is noted that the test results are only an indicator of soil corrosivity, and a corrosion engineer may need to be consulted if the values in Table 5 signify such a need.

9.0 Construction Considerations

It is recommended that the geotechnical investigation report and subsequent addenda be included with project documents during the bidding process for reference purposes.

- Depending on the season of construction, soft, wet surface soils may make it difficult for construction equipment to travel and operate.
- Soils below groundwater level will be wet and unstable, and shallow footings that extend below groundwater level will likely require dewatering and stabilization measures to establish foundation grade. The contractor will be responsible for dewatering design and construction methods.
- Existing underground utilities are present within the project site. The project construction will require coordination of these existing utilities.
- All excavations required on this project should be achievable using typical construction equipment. On-site soils shall be sloped at 1.5H:1V or flatter in temporary excavations (Type C soils). Any excavations below groundwater will require shoring. The contractor will be responsible for design and construction of excavation sloping and shoring in accordance with Cal/OSHA requirements, including the protection of existing structures, utilities and other facilities during construction.

10.0 Quality Control

All plans and specifications should be reviewed by the geotechnical engineer for conformance with this geotechnical report.

The recommendations presented in this report are based on the assumption that sufficient field testing and construction review will be provided during all phases of construction. We should review the final plans and specifications to check for conformance with the intent of our recommendations. Prior to construction, a pre-job conference should be scheduled to include, but not be limited to, the owner, architect, civil engineer, general contractor, earthwork and materials subcontractors, building official, and engineer. The conference will allow parties to review the project plans, specifications, and recommendations presented in this report and discuss applicable material quality and mix design requirements. All quality control reports should be submitted to and reviewed by the engineer.

During construction, we should have the opportunity to provide sufficient on-site observation of preparation and grading, over-excavation, fill placement, foundation installation, and paving. These observations would allow us to verify that the geotechnical conditions are as anticipated and that the contractor's work is in conformance with the approved plans and specifications.

11.0 Standard Limitations Clause

This report has been prepared in accordance with generally accepted geotechnical practices. The analyses and recommendations submitted are based on field exploration performed at the locations shown on Plate 1. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to ensure compliance with our recommendations.

Equilibrium water level readings were made on the date shown on the Boring Logs included as Appendix A-2. Fluctuations in the water table may occur due to rainfall, temperature, seasonal runoff, adjacent irrigation practices, and the water level of the Upper Truckee River. Construction planning should be based on assumptions of possible variations in the water table.

This report has been produced to provide information allowing the architect or engineer to design the project. The client is responsible for distributing this report to all designers and contractors whose work is affected by geotechnical aspects. In the event there are changes in the design, location, or ownership of the project from the time this report is issued, recommendations should be reviewed and possibly modified by the engineer. If the engineer is not granted the opportunity to make this recommended review, he or she can assume no responsibility for misinterpretation or misapplication of his or her recommendations or their validity in the event changes have been made in the original design concept without his or her prior review. The engineer makes no other warranties, either express or implied, as to the professional advice provided under the terms of this agreement and included in this report.

12.0 References

American Association of State Highway and Transportation Officials (AASHTO), 2017, *LRFD Bridge Design Specifications, Customary U.S. Units, 8th Edition.*

American Society for Testing and Materials (ASTM), 2018, *Soil and Rock (I and II),* Volumes 4.08 and 4.09.

Bowles, J. E., 1996, 5th ed., *Foundation Analysis and Design,* McGraw Hill.

California Department of Transportation (Caltrans), 2010, *Soil and Rock Logging, Classification, and Presentation Manual*, 2010 Edition.

Caltrans, 2016, *Geotechnical Manual.*

Caltrans, 2018, *Standard Specifications.*

Caltrans, 2019a, *ARS Online*, Version v2.3.09, accessed June 2019.

Caltrans, 2019b, *Seismic Design Criteria*, Version 2.0, April 2019.

California Geologic Survey (CGS), 2019a, *Fault Activity Map of California*, Online at http://maps.conservation.ca.gov/cgs/fam/, accessed June 2019.

CGS, 2019b, *Earthquake Zones of Required Investigation*, Online at https://maps.conservation.ca.gov/cgs/EQZApp/, accessed June 2019.

Federal Emergency Management Agency (FEMA), 2008 (September 26, 2008), *Flood Insurance Rate Map 06017C0632E, El Dorado County, California, Unincorporated and Incorporated Areas*.

Idriss, I. M., and R. W. Boulanger, 2008, *Soil Liquefaction during Earthquakes*, Earthquake Engineering Research Institute, MNO-12.

Ishihara, K., 1985, *Stability of Natural Deposits During Earthquakes,* Proceedings, 11th Conference on Soil Mechanics and Foundation Engineering, A. A. Balkema, Rotterdam, Netherlands, p. 321-376.

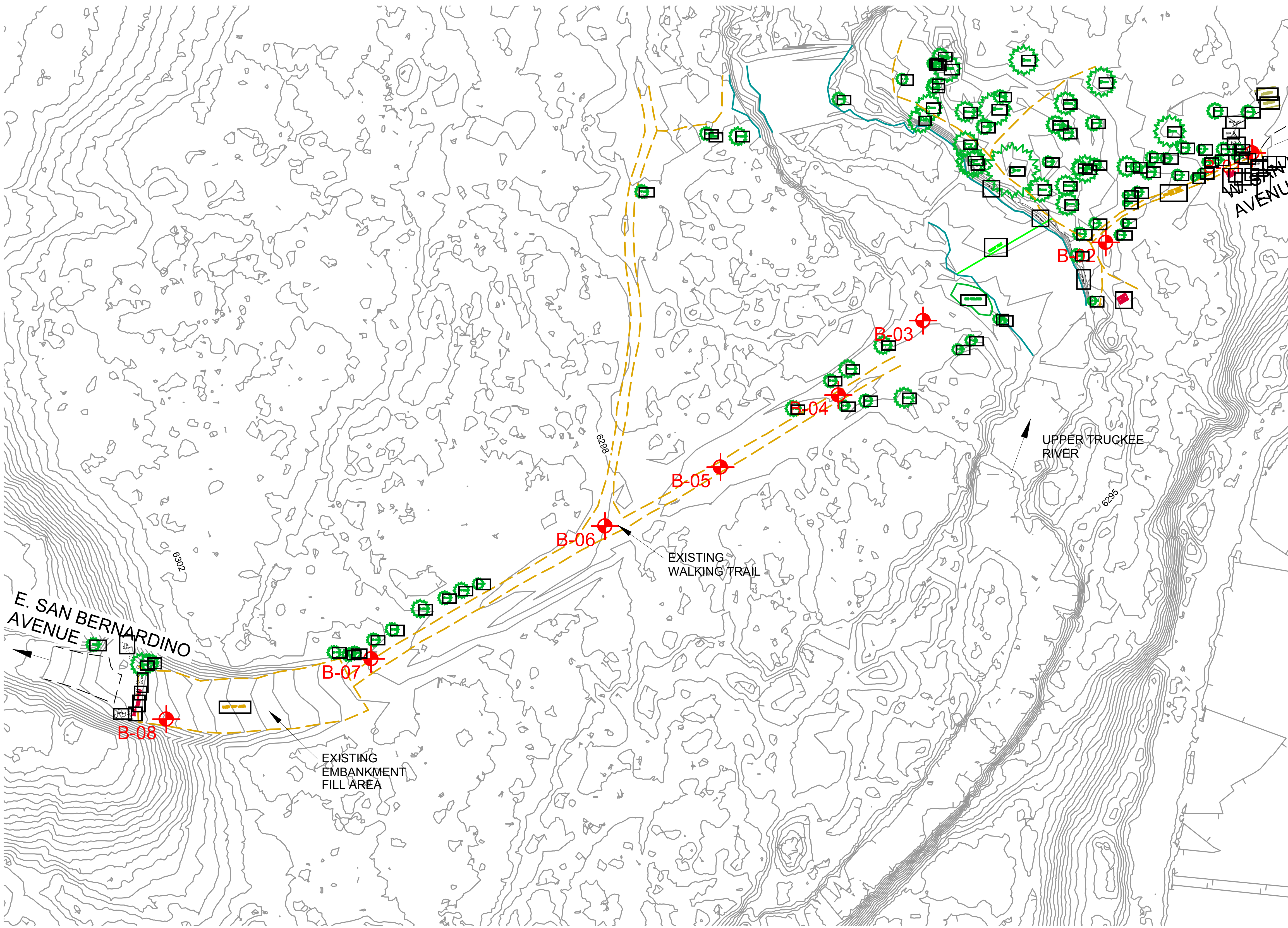
Naval Facilities Engineering Command (NAVFAC), 1986a, *Foundations and Earth Structure;* Design Manual 7.2.

NAVFAC, 1986b, *Soil Mechanics,* Design Manual 7.1.

Saucedo, George J., 2005, *Geologic Map of the Lake Tahoe Basin California and Nevada*, Regional Geologic Map Series, California Geological Survey (CGS), Map No. 4.

United States Geological Survey, *2008 Interactive Deaggregations*, USGS website, http://geohazards.usgs.gov/deaggint/2008/, accessed June 2019.

PLATES



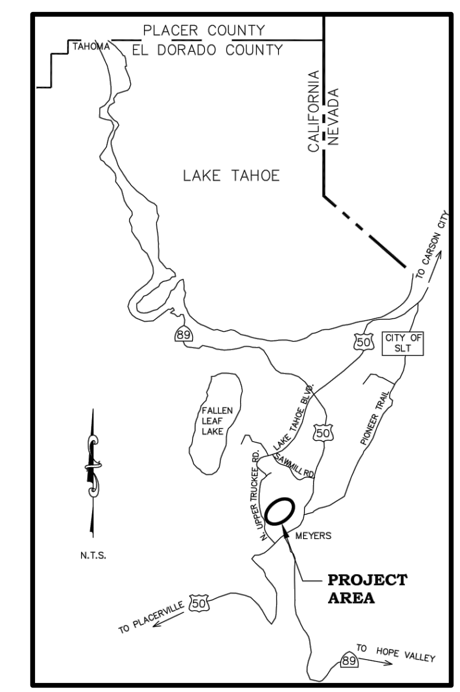
SCALE: 1"=100'
1' CONTOUR INTERVAL

LEGEND

B-01 APPROXIMATE BORING LOCATION

NOTES

1. BASE MAP PROVIDED BY
   EL DORADO COUNTY



VICINITY MAP

N.T.S.

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NCE

LOCATION OF BORINGS MAP

SAN BERNARDINO CLASS 1 BIKE TRAIL PROJECT

EL DORADO COUNTY, CALIFORNIA

Project No.
5012-02-1

Plate 1

SEISMIC DESIGN DATA

San Bernardino Class 1 Bike Trail Project, El Dorado County, California

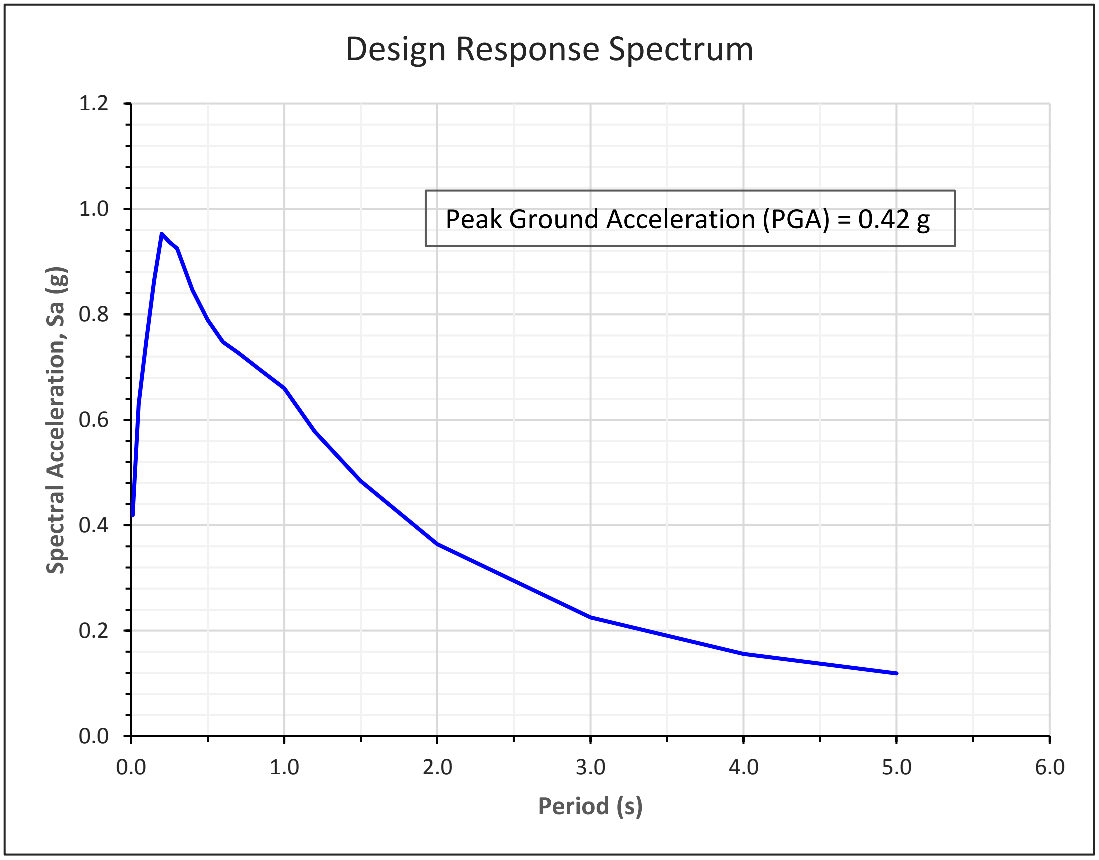
CEI Project No. 5012-02-1

Caltrans ARS Online Version V2.3.09
Accessed Date: June 2019

Site Lattitude:	38.85728
Site Longitude:	-120.02702

Soils Profile:	Class D
Vs30 =	270 m/s

Period (s)	Spectral Acceleration, Sa (g)
0.010	0.419
0.050	0.630
0.100	0.751
0.150	0.863
0.200	0.953
0.250	0.937
0.300	0.925
0.400	0.846
0.500	0.789
0.600	0.748
0.700	0.727
0.850	0.693
1.000	0.660
1.200	0.578
1.500	0.484
2.000	0.364
3.000	0.225
4.000	0.156
5.000	0.119



The Design Response Spectrum is the upper envelope of the deterministic and probablistic response spectrum, but not less than the Minimum Deterministic Spectrum for California.

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APPENDIX A

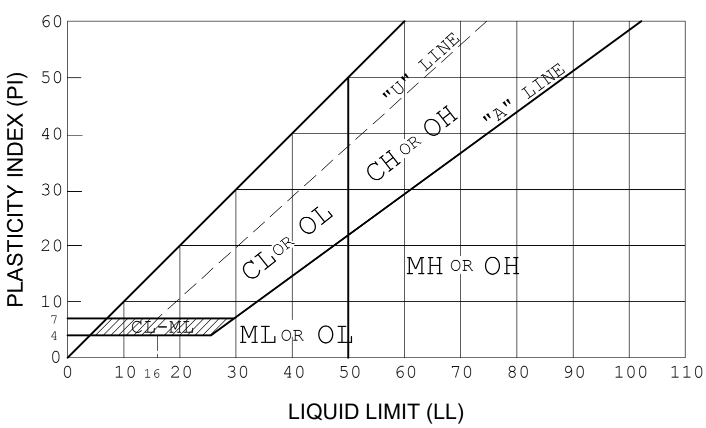
A-1 USCS SOIL CLASSIFICATION CHART
A-2 BORINGS LOGS

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS
FILL MATERIAL				--	FILL MATERIAL, NON-NATIVE

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

PLASTICITY CHART



FOR CLASSIFICATION OF FINE-GRAINED SOILS AND FINE-GRAINED FRACTION OF COARSE-GRAINED SOILS

EXPLORATION SAMPLE TERMINOLOGY

Sample Type	Sample Symbol	Sample Code
Auger Cuttings		Auger
Bulk (Grab) Sample		Grab
Modified California Sampler		MC
Shelby Tube		SH or ST
Standard Penetration Test		SPT
Split Spoon		SS
No Sample		

GRAIN SIZE TERMINOLOGY

Component of Sample	Size Range
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 2mm)
Sand	# 4 to #200 sieve (2mm to 0.074mm)
Silt or Clay	Passing #200 sieve (0.074mm)

RELATIVE DENSITY OF GRANULAR SOILS

N - Blows/ft	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
greater than 50	Very Dense

CONSISTENCY OF COHESIVE SOILS

Unconfined Compressive Strength, psf	N - Blows/ft	Consistency
less than 500	0 - 1	Very Soft
500 - 1,000	2 - 4	Soft
1,000 - 2,000	5 - 8	Firm
2,000 - 4,000	9 - 15	Stiff
4,000 - 8,000	16 - 30	Very Stiff
8,000 - 16,000	31 - 60	Hard
greater than 16,000	greater than 60	Very Hard

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USCS Soil Classification Chart

Project: San Bernardino Class 1 Bike Trail Project

Location: El Dorado County, California

Project Number: 5012-02-1 Plate Number: A-1

BORING LOG

BORING NO.: B-01

DATE: 5/22/2019

TYPE OF BORING: CME 55

DEPTH TO GROUND WATER (ft): 3.7

LOGGED BY: JP

GROUND ELEVATION (ft): 6299±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
	AUGER		12.5	NP		SM		**Silty Sand with Gravel** Brown, moist to wet, medium dense, with 13% non-plastic fines, 69% fine to coarse sand, and 18% subrounded to rounded gravel. Occasional rotten granitic cobbles. Topsoil approximately 4-6 inches thick at exploration location.
A	SPT	21						
B	SPT	29			5			
C	SPT	14				ML		**Silt** Gray, wet, stiff, with an estimated 90% non-plastic fines and 10% fine sand.
D	SPT	21			10	SP-SM		**Poorly Graded Sand with Silt and Gravel** Brown to gray, wet, medium dense, with an estimated 10% non-plastic fines, 70% fine to coarse sand, and 20% subrounded to rounded gravel.
								11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

SHEET 1 OF 1

BORING LOG

BORING NO.: B-02 | DATE: 5/22/2019

TYPE OF BORING: CME 55 | DEPTH TO GROUND WATER (ft): 3.0

LOGGED BY: JP | GROUND ELEVATION (ft): 6296±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	DESCRIPTION
					0–5	SP	**Poorly Graded Sand with Gravel** Brown, moist to wet, medium dense, with an estimated 5% non-plastic fines, 55% fine to coarse sand, and 40% subrounded to rounded, fine to coarse gravel.
A	SPT	30					
B	SPT	7	13.3	NP	5	SP-SM	**Poorly Graded Sand with Silt and Gravel** Brown, wet, loose, with 6% non-plastic fines, 51% fine to coarse sand, and 43% subrounded, fine to coarse gravel.
C	SPT	23				SM	**Silty Sand** Light brown to light gray, wet, medium dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand.
D	SPT	26			10	SM	**Silty Sand with Gravel** Light brown, wet, medium dense, with an estimated 20% non-plastic fines, 65% fine to coarse sand, and 15% subrounded, fine to coarse gravel.
E	SPT	18				SM	**Silty Sand** Light gray, wet, loose to dense, with 19% non-plastic fines, 76% mostly fine to medium sand, and 5% subrounded fine gravel.
F	SPT	10	22.9	NP	15		
G	SPT	24					
H	SPT	33			20		
I	SPT	35				SP-SM	**Poorly Graded Sand with Silt** Brown with orange staining, wet, dense, with an estimated 10% non-plastic fines, 85% fine to coarse sand, and 5% subrounded fine gravel.

Solid-flight auger to 5 feet. Mud-rotary drilling from 5-40 feet depth.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

BORING LOG

BORING NO.: B-02

TYPE OF BORING: CME 55

LOGGED BY: JP

DATE: 5/22/2019

DEPTH TO GROUND WATER (ft): 3.0

GROUND ELEVATION (ft): 6296±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
J	SPT	27				SM		**Silty Sand** Light gray, wet, medium dense, with an estimated 30% non-plastic fines and 70% fine sand.
K	SPT	31			30	SM		**Silty Sand** Light gray, wet, dense, with an estimated 20% non-plastic fines and 80% fine to medium sand.
L	SPT	28			35	SM		**Silty Sand** Light brown to light gray, wet, medium dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand. Includes <1cm thick interbeds of silt (ML). About 2 to 3 per foot.
M	SPT	32			40	SP-SM		**Poorly Graded Sand with Silt** Brown with orange staining, wet, dense, with an estimated 10% non-plastic fines and 90% medium to coarse sand.
					45			41.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger to 5 feet. Mud-rotary drilling from 5-40 feet depth.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

BORING LOG

BORING NO.: B-03

TYPE OF BORING: CME 55

LOGGED BY: JP

DATE: 5/21/2019

DEPTH TO GROUND WATER (ft): 3.0

GROUND ELEVATION (ft): 6295±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	DESCRIPTION
	AUGER		20.5	NP	0–5	SP-SM	**Poorly Graded Sand with Silt** Brown, very moist to wet, loose, with 12% non-plastic fines, 76% fine to coarse sand, and 12% subangular to subrounded, fine to coarse gravel.
A	SPT	4					
B	MC	32	11.7	NP	5–7.5	SP-SM	**Poorly Graded Sand with Silt and Gravel** Brown, wet, medium dense, with 6% non-plastic fines, 69% fine to coarse sand, and 25% subangular to subrounded, fine to coarse gravel.
C	SPT	19			7.5–10	SM	**Silty Sand with Gravel** Brown with orange mottling, wet, medium dense, with an estimated 15% non-plastic fines, 55% fine to coarse sand, and 30% subrounded to rounded gravel up to 1 inch in diameter.
D	MC	44	11.3	NP	10–12.5	SW-SM	**Well-Graded Sand with Silt and Gravel** Brown, wet, medium dense, with 8% non-plastic fines, 72% fine to coarse sand, and 20% subrounded gravel up to 1 inch in diameter.
E	SPT	39			12.5–17	SM	**Silty Sand** Light brown to light gray, wet, medium dense to dense, with an estimated 20% non-plastic fines, 75% fine to medium sand, and 5% subrounded gravle up to 1 inch in diameter. 15 feet below the ground surface (bgs): switch to HQ coring due to hole collapse.
F	SPT	24					
G	SPT	32			17–		**Silty Sand** Light gray, wet, medium dense to dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand.
H	SPT	29					
I	SPT	34					

Solid-flight auger to 15 feet. HQ core drilling from 15-40 feet depth.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

CEI_BORING LOG 17020115012021.GPJ BLKEAGLE.GDT 7/26/2019

BORING LOG

BORING NO.: B-03

TYPE OF BORING: CME 55

LOGGED BY: JP

DATE: 5/21/2019

DEPTH TO GROUND WATER (ft): 3.0

GROUND ELEVATION (ft): 6295±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
J	SPT	40				SM		
K	SPT	34						
					30			
L	SPT	26						
M	SPT	26						
					35			
N	SPT	24				SM		**Silty Sand** Light gray, wet, medium dense, with an estimated 25% non-plastic fines and 75% fine to medium sand.
					40			
O	SPT	34				SM		**Silty Sand** Light gray, wet, dense, with an estimated 15% non-plastic fines and 85% fine to coarse sand.
								41.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.
					45			

Solid-flight auger to 15 feet. HQ core drilling from 15-40 feet depth.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2a

SHEET 2 OF 2

CEI_BORING LOG 17020115012021.GPJ BLKEAGLE.GDT 7/26/2019

BORING LOG

BORING NO.: B-04
TYPE OF BORING: CME 55
LOGGED BY: JP

DATE: 5/21/2019
DEPTH TO GROUND WATER (ft): 2.9
GROUND ELEVATION (ft): 6296±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
					0–5	SM		**Silty Sand with Gravel** Brown, very moist to wet, medium dense, with an estimated 20% non-plastic fines, 65% fine to coarse sand, and 15% subrounded to rounded gravel up to 1 inch in diameter. Topsoil approximately 2-4 inches thick at exploration location.
A	SPT	17						
B	SPT	22			5	SP-SM		**Poorly Graded Sand with Silt and Gravel** Brown to orange brown, moist to wet, medium dense, with an estimated 10% non-plastic fines, 60% fine to coarse sand, and 30% subrounded to rounded gravel up to 3/4 inch in diameter.
C	SPT	20				SM		**Silty Sand** Brown to light gray, wet, medium dense, with an estimated 20% non-plastic fines, 70% fine to medium sand, and 10% subrounded gravel up to 1/2 inch in diameter.
D	SPT	19			10	SP		**Poorly Graded Sand with Gravel** Orange brown, wet, medium dense, with an estimated 5% non-plastic fines, 60% fine to coarse sand, and 35% subrounded gravel up to 1 inch in diameter. Heavy soil staining.
								11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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NCE
San Bernardino Class 1 Bike Trail Project
El Dorado County, California

PROJECT NO.: 5012-02-1
PLATE: A-2
SHEET 1 OF 1

BORING LOG

BORING NO.: B-05

TYPE OF BORING: CME 55

LOGGED BY: JP

DATE: 5/21/2019

DEPTH TO GROUND WATER (ft): 1.5

GROUND ELEVATION (ft): 6297±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
						SP		**Poorly Graded Sand with Gravel** Brown to orange brown, moist to wet, medium dense, with an estimated 5% non-plastic fines, 60% fine to coarse sand, and 35% subrounded, fine to coarse gravel. Topsoil approximately 2-4 inches thick at exploration location. Hard drilling from 3-4 feet bgs.
A	SPT	20						
B	SPT	19			5	SM		**Silty Sand with Gravel** Brown, wet, medium dense, with an estimated 20% non-plastic fines, 60% fine to coarse sand, and 20% subangular to subrounded, fine to coarse gravel. Hard drilling from 3-4 feet bgs.
C	SPT	24				SP-SM		**Poorly Graded Sand with Silt and Gravel** Orange brown, wet, medium dense, with an estimated 10% non-plastic fines, 70% fine to coarse sand, and 20% subrounded gravel up to 1 inch in diameter. Heavy soil staining.
D	SPT	28			10	SM		**Silty Sand** Light gray, wet, medium dense, with an estimated 30% non-plastic fines and 70% fine sand.
								11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

BORING LOG

BORING NO.: B-06

DATE: 5/22/2019

TYPE OF BORING: CME 55

DEPTH TO GROUND WATER (ft): 0.75

LOGGED BY: JP

GROUND ELEVATION (ft): 6298±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	DESCRIPTION
					0–5	SP	**Poorly Graded Sand with Gravel** Brown to orange brown, moist to wet, loose, with an estimated 5% non-plastic fines, 75% fine to coarse sand, and 20% subrounded gravel up to 1 inch in diameter. Topsoil approximately 2-4 inches thick at exploration location.
A	SPT	6					
B	SPT	21			5	SM	**Silty Sand with Gravel** Brown, wet, medium dense, with an estimated 15% non-plastic fines, 55% fine to coarse sand, and 30% subrounded gravel up to 1 inch in diameter.
C	SPT	11				SM	**Silty Sand** Light gray, wet, medium dense, with an estimated 25% non-plastic fines, 75% fine to medium sand, and trace amounts of subrounded, fine gravel.
D	SPT	10			10	SM	**Silty Sand** Light gray, wet, loose, with an estimated 20% non-plastic fines and 80% fine to coarse sand.
							11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

SHEET 1 OF 1

BORING LOG

BORING NO.: B-07
TYPE OF BORING: CME 55
LOGGED BY: JP

DATE: 5/22/2019
DEPTH TO GROUND WATER (ft): 1.0
GROUND ELEVATION (ft): 6301±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
	AUGER		22.3	NP		SM		**Silty Sand with Gravel** Brown, moist to wet, medium dense, with 20% non-plastic fines, 59% fine to coarse sand, and 21% subrounded to rounded, fine to coarse gravel. Trace amounts of cobbles up to 6 inches in diameter. Topsoil approximately 2-4 inches thick at exploration location.
A	SPT	23						
B	SPT	17			5	SM		**Silty Sand** Light gray, wet, medium dense, with an estimated 30% non-plastic fines and 70% fine sand.
					10			11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

PROJECT NO.: 5012-02-1

PLATE: A-2

SHEET 1 OF 1

CEI_BORING LOG 17020115012021.GPJ BLKEAGLE.GDT 7/26/2019

BORING LOG

BORING NO.: B-08

TYPE OF BORING: CME 55

LOGGED BY: JP

DATE: 5/22/2019

DEPTH TO GROUND WATER (ft): 7.1

GROUND ELEVATION (ft): 6311±

SAMPLE NO.	SAMPLE TYPE	BLOWS/12 inches	MOISTURE (%)	PLASTICITY INDEX	DEPTH (ft)	USCS SYMBOL	LITHOLOGY	DESCRIPTION
								Poorly Graded Sand with Silt and Gravel (Fill) Brown and gray, moist to wet, loose to medium dense, with 8% non-plastic fines, 55% fine to coarse sand, and 37% subangular to subrounded, fine to coarse gravel.
A	SPT	20	6.6	NP		SP-SM		
B	SPT	9			5			
					▼			
C	SPT	2				SM		**Silty Sand** Brown to gray, wet, very loose, with an estimated 35% non-plastic fines, 55% fine sand, and 10% subangular to subrounded, fine gravel.
D	SPT	37			10	SP-SM		**Poorly Graded Sand with Silt and Gravel** Orange brown, wet, dense, with an estimated 10% non-plastic fines, 55% fine to coarse sand, and 35% subrounded to rounded, fine to coarse gravel.
								11.5 feet total depth, terminated at planned depth. Backfilled with neat cement grout.

Solid-flight auger drilling.

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San Bernardino Class 1 Bike Trail Project

El Dorado County, California

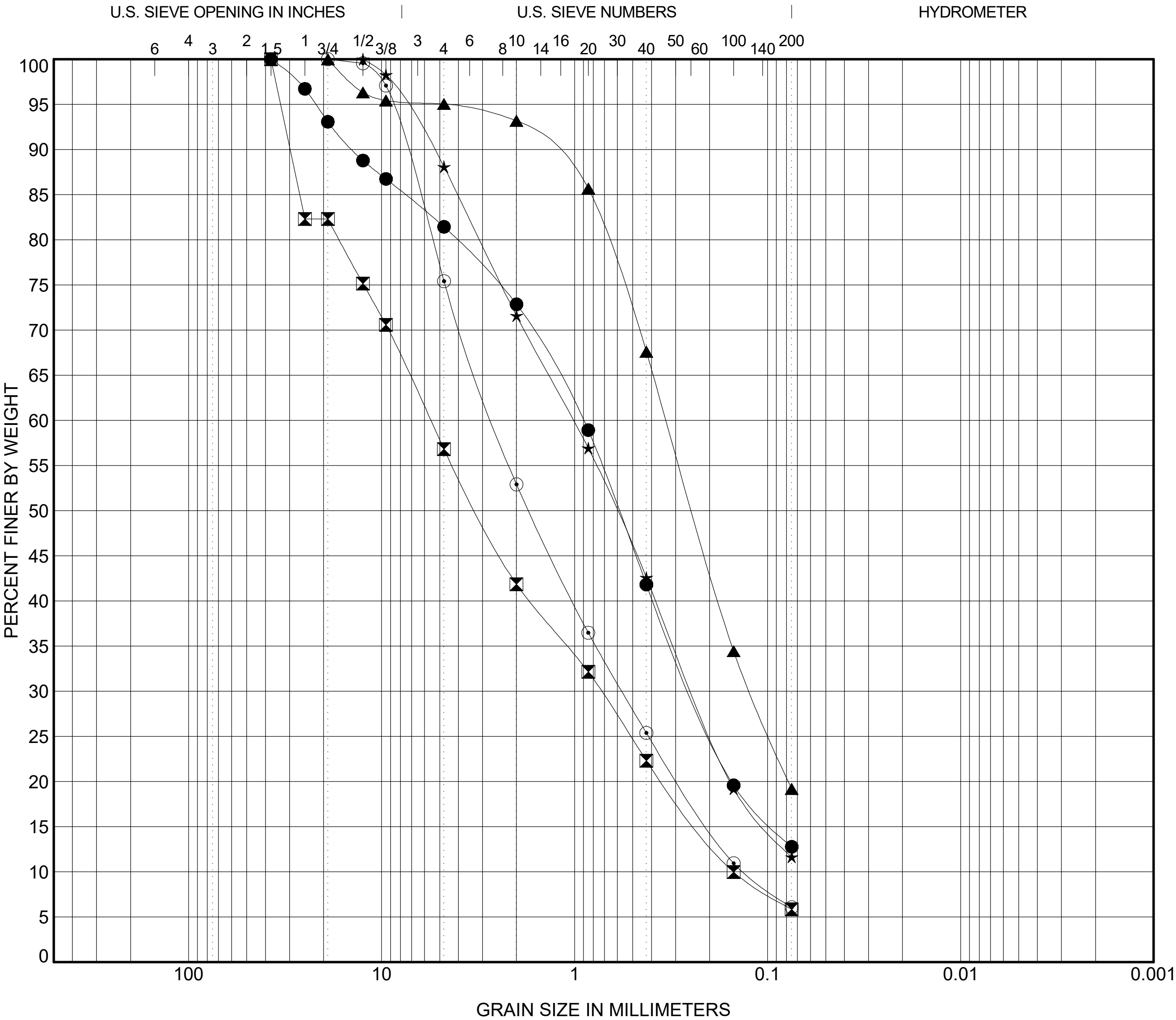
PROJECT NO.: 5012-02-1

PLATE: A-2

SHEET 1 OF 1

APPENDIX B

B-1 INDEX TEST RESULTS
B-2 DIRECT SHEAR TEST RESULTS
B-3 R-VALUE TEST RESULTS
B-4 CHEMICAL TEST RESULTS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**0.0'**	**SILTY SAND with GRAVEL (SM)**	**NP**	**NP**	**NP**		
⧗	**B-02**	**5.0'**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**	**NP**	**NP**	**NP**	**0.64**	**37.10**
▲	**B-02**	**12.5'**	**SILTY SAND (SM)**	**NP**	**NP**	**NP**		
★	**B-03**	**0.0'**	**POORLY GRADED SAND with SILT (SP-SM)**	**NP**	**NP**	**NP**	**0.90**	**15.77**
⊙	**B-03**	**5.0'**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**	**NP**	**NP**	**NP**	**0.93**	**20.03**

	Specimen Identification		D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt / %Clay
●	**B-01**	**0.0'**	**37.5**	**0.908**	**0.244**		**12.5**	**18.6**	**68.7**	**12.8**
⧗	**B-02**	**5.0'**	**37.5**	**5.578**	**0.731**	**0.15**	**13.3**	**43.2**	**51.0**	**5.8**
▲	**B-02**	**12.5'**	**19**	**0.335**	**0.123**		**22.9**	**5.0**	**75.9**	**19.2**
★	**B-03**	**0.0'**	**12.5**	**1.017**	**0.243**		**20.8**	**11.9**	**76.4**	**11.6**
⊙	**B-03**	**5.0'**	**19**	**2.627**	**0.567**	**0.131**	**11.7**	**24.6**	**69.4**	**6.1**

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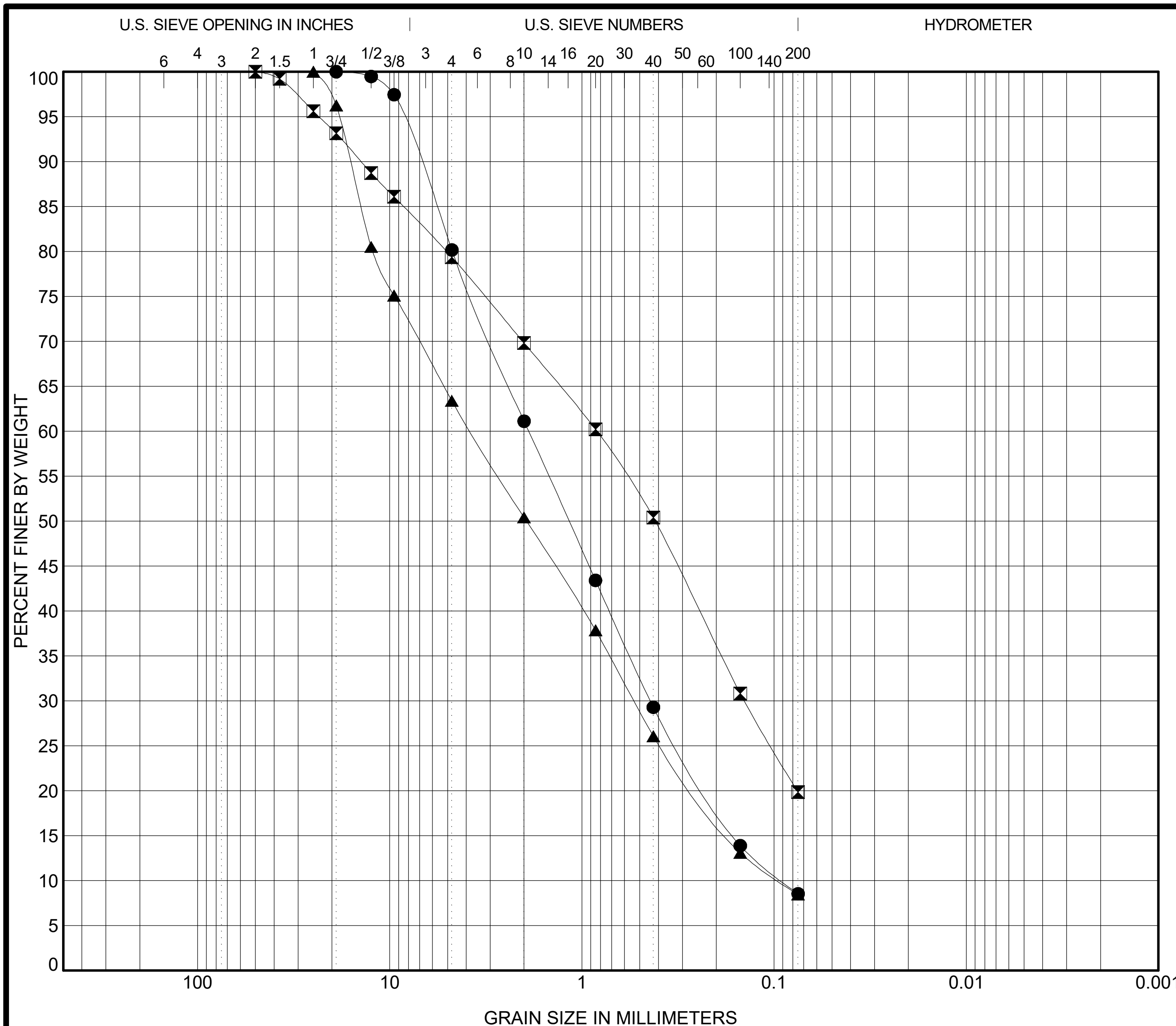
GRAIN SIZE DISTRIBUTION

Project: San Bernardino Class 1 Bike Trail Project

Location: El Dorado County, California

Project Number: 5012-02-1 Plate Number: B-1.a



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		USCS Classification	LL	PL	PI	Cc	Cu
●	**B-03**	**10.0'**	**WELL-GRADED SAND with SILT and GRAVEL (SW-SM)**	**NP**	**NP**	**NP**	**1.13**	**20.89**
⧗	**B-07**	**0.0'**	**SILTY SAND with GRAVEL (SM)**	**NP**	**NP**	**NP**		
▲	**B-08**	**2.5'**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**	**NP**	**NP**	**NP**	**0.80**	**40.10**

	Specimen Identification		D100	D60	D30	D10	MC %	%Gravel	%Sand	%Silt	%Clay
●	**B-03**	**10.0'**	**19**	**1.895**	**0.44**	**0.091**	**11.3**	**19.8**	**71.6**	**8.5**	
⧗	**B-07**	**0.0'**	**50**	**0.837**	**0.142**		**22.3**	**20.6**	**59.5**	**19.8**	
▲	**B-08**	**2.5'**	**25**	**3.792**	**0.536**	**0.095**	**6.9**	**36.6**	**54.9**	**8.5**	

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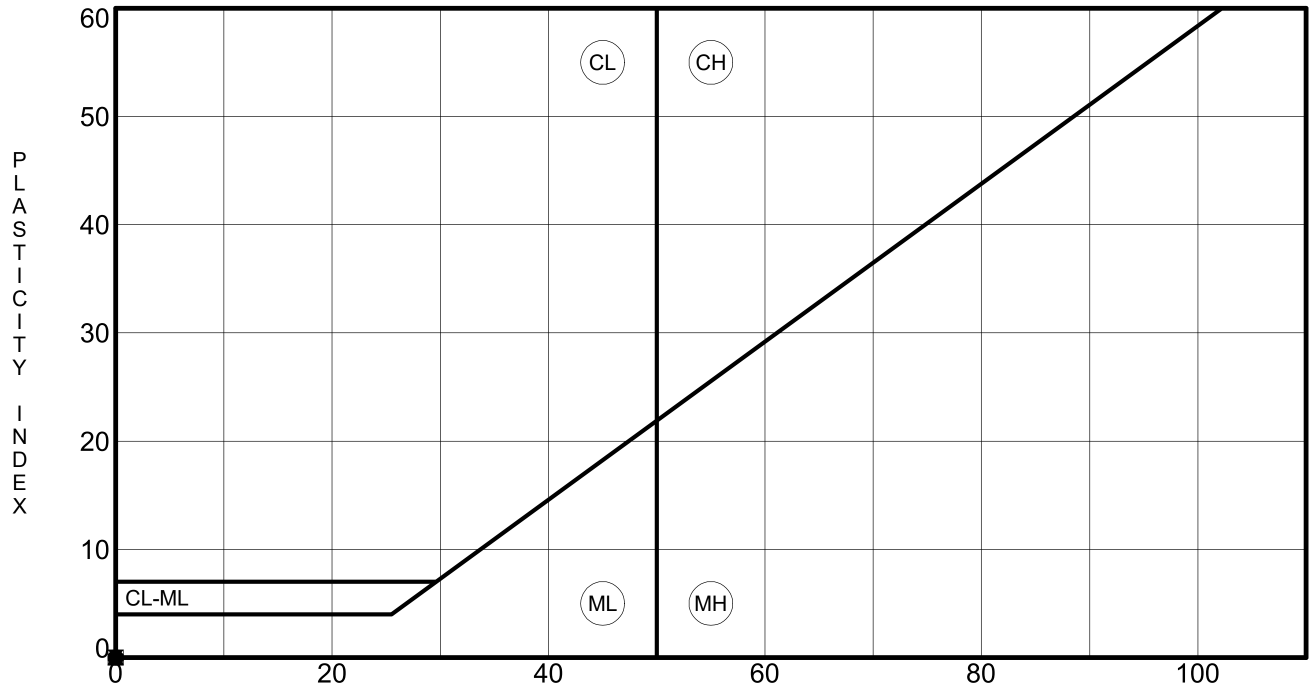
GRAIN SIZE DISTRIBUTION

Project: San Bernardino Class 1 Bike Trail Project

Location: El Dorado County, California

Project Number: 5012-02-1 Plate Number: B-1.b



Specimen Depth in Feet.

	Specimen Identification		LL	PL	PI	Fines	USCS Classification
●	**B-01**	**0.0'**	**NP**	**NP**	**NP**	**13**	**SILTY SAND with GRAVEL (SM)**
⧗	**B-02**	**5.0'**	**NP**	**NP**	**NP**	**6**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**
▲	**B-02**	**12.5'**	**NP**	**NP**	**NP**	**19**	**SILTY SAND (SM)**
★	**B-03**	**0.0'**	**NP**	**NP**	**NP**	**12**	**POORLY GRADED SAND with SILT (SP-SM)**
⊙	**B-03**	**5.0'**	**NP**	**NP**	**NP**	**6**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**
✿	**B-03**	**10.0'**	**NP**	**NP**	**NP**	**9**	**WELL-GRADED SAND with SILT and GRAVEL (SW-SM)**
○	**B-07**	**0.0'**	**NP**	**NP**	**NP**	**20**	**SILTY SAND with GRAVEL (SM)**
△	**B-08**	**2.5'**	**NP**	**NP**	**NP**	**8**	**POORLY GRADED SAND with SILT and GRAVEL (SP-SM)**

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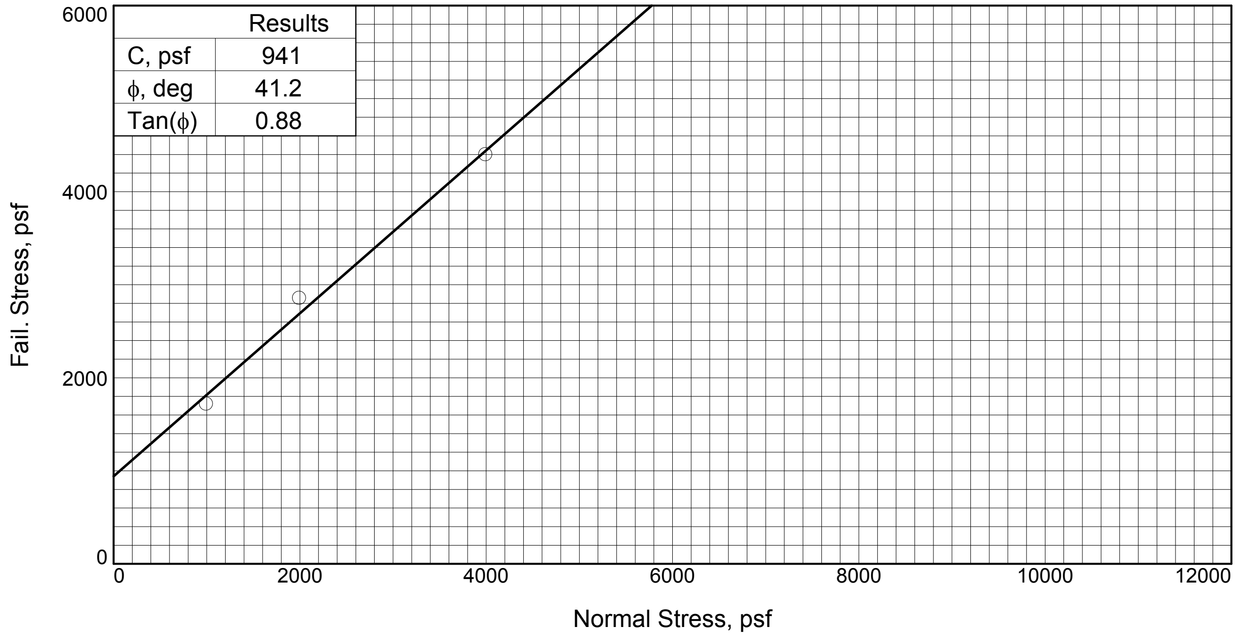
ATTERBERG LIMITS RESULTS

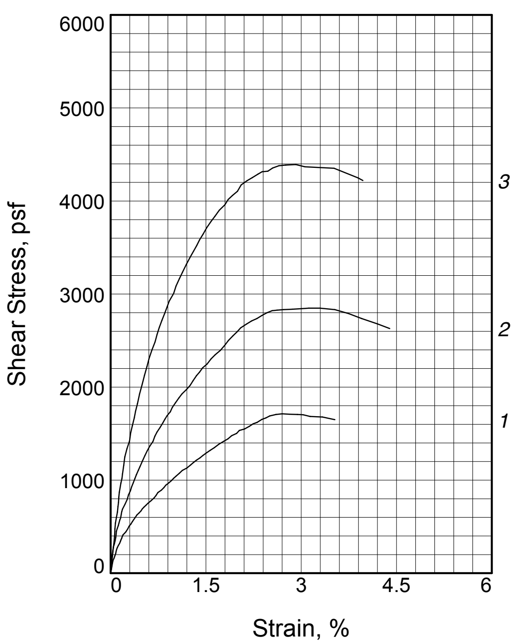
Project: San Bernardino Class 1 Bike Trail Project

Location: El Dorado County, California

Project Number: 5012-02-1 Plate Number: B-1.c



	Results
C, psf	941
ϕ, deg	41.2
Tan(ϕ)	0.88



	Sample No.	1	2	3
Initial	Water Content, %	11.8	11.8	11.8
	Dry Density, pcf	122.8	122.8	122.8
	Saturation, %	85.4	85.4	85.4
	Void Ratio	0.3721	0.3721	0.3721
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	13.8	12.0	13.0
	Dry Density, pcf	122.9	127.3	124.9
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.3715	0.3243	0.3500
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	0.97	0.98
	Normal Stress, psf	1000	2000	4000
	Fail. Stress, psf	1712	2849	4392
	Strain, %	2.7	3.1	2.9
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Remolded to In-Situ Density
Description: Poorly Graded Sand with Silt and Gravel
LL= 0 **PI=** NP
Assumed Specific Gravity= 2.7
Remarks: Laboratory Log 7434

Figure B-2.a

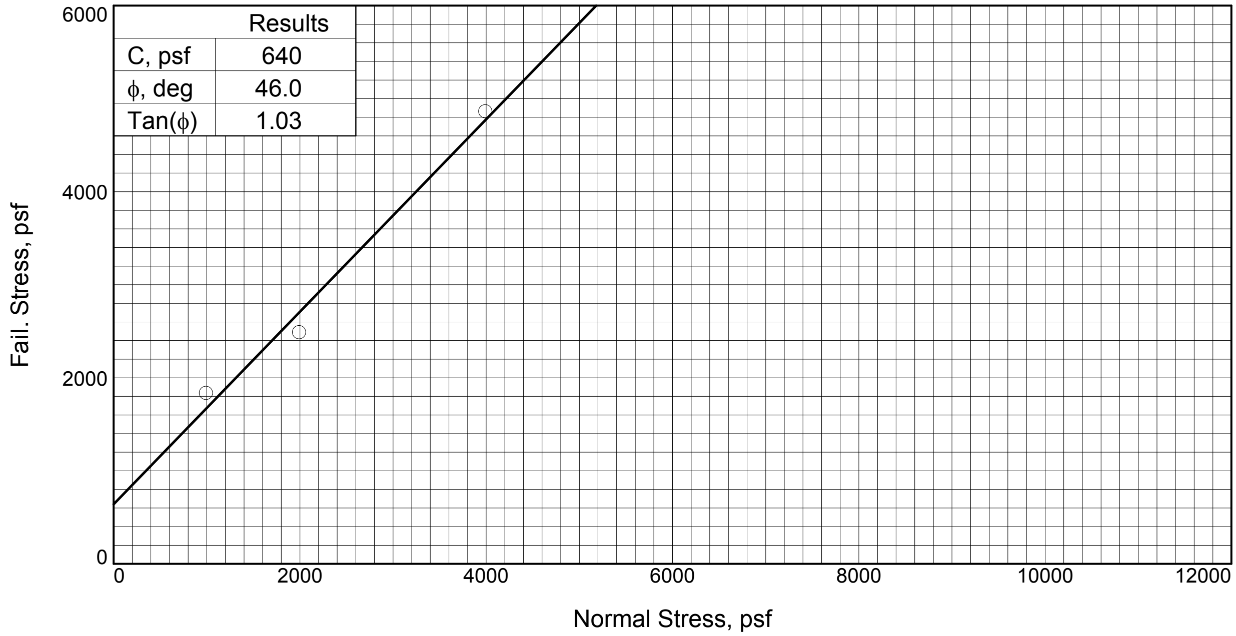
Client: Corestone Engineering, Inc.

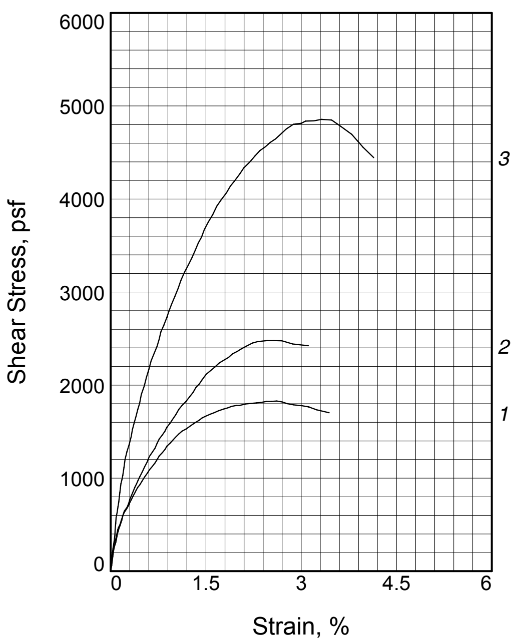
Project: San Bernardino Class 1 Bike Trail Project

Source of Sample: B-03 **Depth:** 5
Sample Number: B
Proj. No.: 1702-01-1 **Date Sampled:** 05/21/19

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: GLO **Checked By:** LO





Sample No.		1	2	3
Initial	Water Content, %	11.3	11.3	11.3
	Dry Density, pcf	123.8	123.8	123.8
	Saturation, %	84.4	84.4	84.4
	Void Ratio	0.3614	0.3614	0.3614
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	12.6	10.7	13.3
	Dry Density, pcf	125.6	130.7	124.0
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.3415	0.2899	0.3597
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.95	1.00
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		1828	2480	4856
Strain, %		2.6	2.5	3.3
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Remolded to In-Situ Density
Description: Well-Graded Sand with Silt and Gravel

LL= 0 **PI=** NP
Assumed Specific Gravity= 2.7
Remarks: Laboratory Log 7434

Figure B-2.b

Client: Corestone Engineering, Inc.

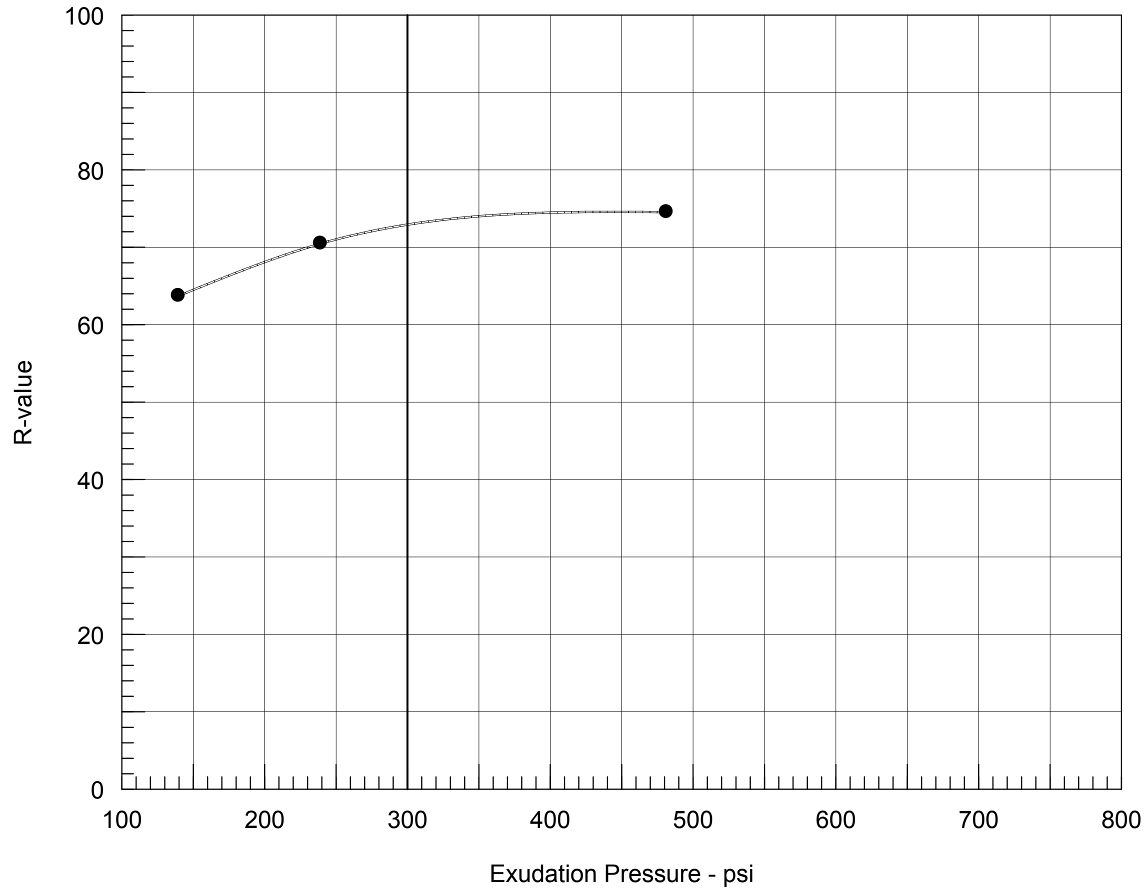
Project: San Bernardino Class 1 Bike Trail Project

Source of Sample: B-03 **Depth:** 10
Sample Number: D
Proj. No.: 1702-01-1 **Date Sampled:** 05/21/19

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: GLO **Checked By:** LO

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	250	104.8	17.5	0.06	47	2.63	140	61	64
2	300	105.1	16.7	0.58	35	2.52	239	70	70
3	350	106.1	15.1	0.82	30	2.52	482	75	75

Test Results	Material Description
R-value at 300 psi exudation pressure = 73	Silty Sand with Gravel

Project No.: 1702-01-1

Project: San Bernardino Class 1 Bike Trail Project

Source of Sample: B-01 **Depth:** 0

Sample Number: Bulk

Date: 7/26/2019

Tested by: GLO
Checked by: LO

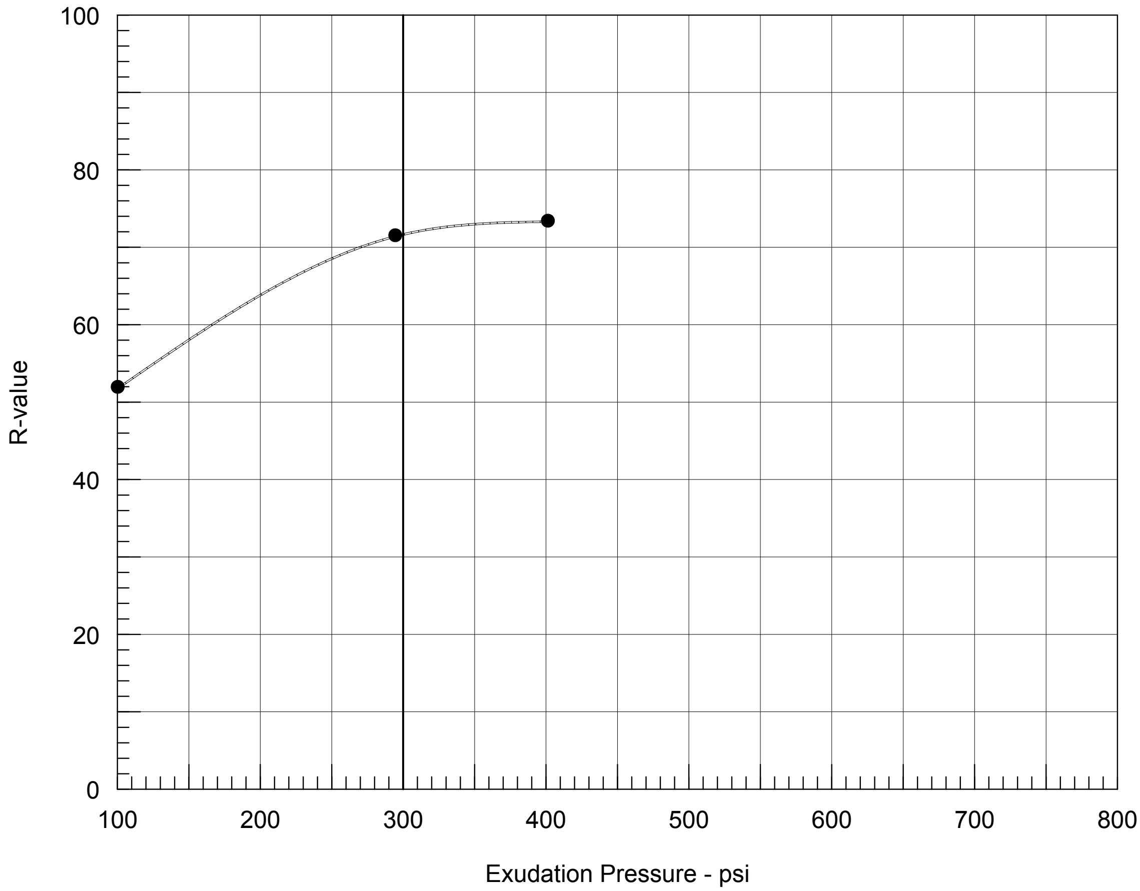
Remarks:
Laboratory Log 7434

R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Figure B-3.a

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	280	111.3	15.3	0.06	35	2.48	295	71	71
2	350	111.5	13.9	0.00	32	2.54	402	73	73
3	200	110.7	15.9	0.36	60	2.45	101	52	52

Test Results	Material Description
R-value at 300 psi exudation pressure = 72	Silty Sand with Gravel

Project No.: 1702-01-1

Project: San Bernardino Class 1 Bike Trail Project

Source of Sample: B-07 **Depth:** 0

Sample Number: Bulk

Date: 7/26/2019

Tested by: GLO

Checked by: LO

Remarks:
Laboratory Log 7434

R-VALUE TEST REPORT

BLACK EAGLE CONSULTING, INC.

Figure B-3.b

APPENDIX C

Project Name:	San Bernardino Class 1 Bike Trail Project	Developed By:	PV
Project No:	5012-02-1	Calculated By:	JP/PV
Boring No:	B-02 MR Boring	Checked By:	PV
Analyzed Case:	Bridge	Date:	6/26/2019
	Selected potential liquefiable layers	Version:	Jan-14

Liquefaction Potential of a Single Layer Using Idriss and Boulanger (2008) SPT Method

Input Parameters

Earthquake Input Parameters

Peak Ground Acceleration, a_{max} =	0.42	g	S_{DS}/2.5 or PGA
Earthquake Magnitude, M =	6.48		USGS Deaggregation Analysis
		(or known/active nearby fault's $M_{probable}$)	

Layer and SPT Test Data

Depth to Layer Top =	17.5	feet	
Thickness of the Layer =	2.5	feet	
SPT Sample Depth =	17.5	feet	
Measured SPT N-Value =	24		
Depth to Ground Water Table =	2.5	feet	Design Value - Measured 3'
Hammer Energy Efficiency ER =	75	%	Auto hammer (Taber)
Borehole Diameter =	4	inch	101.6 mm
Standard SPT Sampler? (Yes/No)	Yes	(Yes: 1-3/8" inside dia - No room for liner)	

Soil Parameters

USCS Soil Type =	SM		
% Fines =	19	%	Non-Liq if >35% & PI >7
Plasticity Index, PI =	NP	(Info only)	
Average unit weight above GW =	120	pcf	
Average unit weight below GW =	120	pcf	

Void redistribution effect? (Yes/No) No (Only for shear strength calcs)

(Select yes only for thick liquefiable layer that is underlain by low pearmeable deposists)

Caculations

Total and Efeective Stress

Mid depth to SPT sample, z =	18.5	feet	5.6388 m
Total Stress at Mid Depth, σ_{vo} =	2220	psf	
Effective Strees at Mid Depth σ_{v0}' =	1222	psf	

SPT Corrections

C_E = 1.250 C_B = 1.00 C_S = 1.00 C_R = 0.95

C_N = 1.188 or 1.316 (alternative equation)

$(N_1)_{60}$ =	33.9	37.5	corrected SPT blow count
ΔN =	4.3		correction for percent of fines (add)
$(N_1)_{60\ CS}$ =	38.2		Clean-sand equivalent corrected SPT blow count

Cyclic Stress Ratio

rd =	0.920	stress reduction coefficient
CSR =	0.456	Cyclic stress ratio for design EQ

Cyclic Resistance Ratio

$CRR_{M7.5,\,1}$ =	2.000	Cyclic resistance ratio for M=7.5 & σ_{vc}' =1atm
MSF =	1.308	EQ magnitude scaling factor
K_{σ} =	1.100	Overburden correction factor
CRR =	2.877	Cyclic resistance ratio for M & σ_{v0}'

Factor of Safety

FS_{liq} =	**2.000**	Factor of Safety Against Soil Liquefaction

Limit maximum to 2.0 (for plotting purpose)

Lateral Spread

γ_{lim} =	1.3%	Limiting shear strain
F_{α} =	-0.666	Parameter F_{α}
γ_{max} =	0.0%	Maximum shear strain
LDI =	0.000 feet	Lateral displacement index (displacement in the subject layer)

1-D Reconsolidation Settlement (Liquefaction Induced Vertical Settlement)

εv =	0.00%	volumetric strain
S =	0.00 inches	Liquefaction vertical settlement (at the considered layer)

Residual Shear Strength

$\Delta(N_1)_{60\text{-}Sr}$ =	1.6	Fine correction for residual strength by Seed (1987)
$(N_1)_{60\ CS\text{-}Sr}$ =	35.4	Clean-sand equivalent SPT blow count for S_r
S_r/σ_{v0}' =	0.400	Residual Shear Strength Ratio
S_r =	490 psf	Residual Shear Strength

Results Summary:

Boring	Top Depth (feet)	Thickness (feet)	USCS Type	N	$(N_1)_{60\,CS}$	CSR	CRR	FS_{liq}	LDI (feet)	S (inches)	S_r (psf)
B-02	17.5	2.5	SM	24	38.2	0.456	2.877	2.000	0.000	0.00	490

Notes:

1. FS_{liq} - Factor of safety with respect to soil liquefaction; <1.0 potential exists, <1.1 marginal
2. LDI -Lateral spread index/displacement. If the liquefiable layer is at a depth deeper than twise the vertical height of the free-face, potential for lateral spread would be minimal (for free-face height of less than 10 feet).
3. S - Liquefaction induced vertical settlement at the layer. Surface manifestaion would be smaller and will depend on the thickness of the non-liquefiable cap above.
4. S_r - Estimated residual strength of the liquefied soils.

Saved Results:

Boring	Top Depth (feet)	Thickness (feet)	USCS Soil	N	$(N_1)_{60\,CS}$	CSR	CRR	FS_{liq}	LDI (feet)	S (inches)	S_r (psf)
B-02	5.0	2.5	SP-SM	7	11.9	0.386	0.190	0.492	0.958	1.01	70
B-02	12.5	2.5	SM	18	34.5	0.386	1.436	2.000			
B-02	15.0	2.5	SM	10	20.6	0.454	0.305	0.671	0.322	0.67	430
B-02	17.5	2.5	SM	24	38.2	0.456	2.877	2.000			
								Σ	1.280	1.68	

PW: liq

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Date: 6/25/2019
Revision Number: 0319
Developed By: JWP/PV
Calculated By: PV
Checked By:

Project Name: San Bernardino Class 1 Bike Trail Project
Project Number: 5012-02-1

CALCULATION OF LRFD 8TH EDITION (2017) BEARING CAPACITY

Location: Pier Footings on Native Ground
Foundation: 5 feet Wide Footing footing

References

1. AASHTO, 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, American Association of State Highway and Transportation Officials.

Assumptions

1. Bearing capacity calculations account for foundation shape, possibility of local or punching shear, inclined load, eccentric loading, sloping ground, and ground water.
2. Calculations assume one, homogeneous soil unit. Two-layer soil systems not supported.

Unit Conversions **Checked By:**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $g = 32.174\frac{ft}{s^2}$ $ksf := \frac{kip}{ft^2}$ $kPa := 1000Pa$ $kN := 1000N$ $kJ := 1000J$

Input Data **Checked By:**

Soil Cohesion:	$c := 0psf$	$c = 0.0 \cdot kPa$
Soil Friction Angle:	$\phi := 36deg$	
Total Soil Unit Weight:	$\gamma := 20\frac{kN}{m^3}$	$\gamma = 127.3 \cdot pcf$
Depth of Foundation Base below Ground Surface:	$D_f := 0m$	$D_f = 0.00 \cdot ft$
Foundation Width B (For Circular Footings B = L):	$B := 1.524m$	$B = 5.00 \cdot ft$
Foundation Length L:	$L := 5.4864m$	$L = 18.00 \cdot ft$
Depth of Ground Water from Ground Surface:	$D_w := 0ft$	$D_w = 0.00$
Slope of Adjacent Ground (if j>0, the modified Nγ and Nc apply below, Nq=0):	$j := 0deg$	
Calculate estimate reduction factor from Table 10.6.3.1.2c-1 or -2 and calculate teh reduced bearing capacity factors	$N_{\gamma slope} := 19$ $N_{cslope} := 0$	for β = 20.6 deg.
Is Local or Punching Shear Possible (Yes = "Y" and No = "N")?	$F_{ps} := \text{"N"}$	
Unfactored Vertical Load on Footing (Vertical):	$V := 1500kN$	$V = 337.2 \cdot kip$
Unfactored Horiz Load on Footing (Enter 0 for vertical load only):	$H := 0kip$	$H = 0.0 \cdot kN$
Orientation of Horizontal Load (Enter 0 for parallel to long axis L):	$\theta := 0deg$	
Moment in x-Dimension (Footing Width):	$M_x := 0kip \cdot ft$	$M_x = 0.0 \cdot kJ$
Moment in y-Dimension (Footing Length):	$M_y := 0kip \cdot ft$	$M_y = 0.0 \cdot kJ$
Adhesion Between Footing and Foundation Soil for Sliding:	$c_a := 0psf$	$c_a = 0.0 \cdot kPa$
Angle of Friction Between Footing and Foundation Soil for Sliding:	$\delta := 0.8 \cdot \phi$	$\delta = 28.8 \cdot deg$

Sliding Resistance Factor for the Strength Limit State: $\phi_\tau := 0.80$ CIP on sand

Bearing Resistance Factor for the Strenght Limit State: $\phi_b := 0.45$ This is a the Munfakh (2001) approach, ϕb varies from 0.45 to 0.5
Bearing Resistance Factor for Extreme State(scour, EQ, ice, impacts = 1.0)
Bearing Resistance Factor for Service State (Settlements and Servicability = 1.0)

An exception for service limit state 1 is that overall stability shall use resistance factors in Article 11.6.2.3

Calculations, Section 1: Bearing Pressures, Eccentricity Reduction **Checked By:**

Calculate Eccentricity in Footing "B" Direction:

$e_B := \dfrac{M_y}{V}$ $\qquad e_B = 0.0 \cdot ft \qquad e_B = 0.00$

Calculate Eccentricity in Footing "L" Direction:

$e_L := \dfrac{M_x}{V}$ $\qquad e_L = 0.0 \cdot ft \qquad e_L = 0.00$

Calculate Eccentric Loading Reduced Footing Dimensions:

$B' := B - 2 \cdot e_B$ $\qquad B' = 5.0 \cdot ft \qquad B' = 1.52\,m$

$L' := L - 2 \cdot e_L$ $\qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$

Determine Effective Footing Dimensions based on any Eccentricity:

$B' := \begin{cases} B' & \text{if } e_B > 0ft \\ B & \text{otherwise} \end{cases}$ $\qquad B' = 5.0 \cdot ft \qquad B' = 1.52\,m$

$L' := \begin{cases} L' & \text{if } e_L > 0ft \\ L' & \text{otherwise} \end{cases}$ $\qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$

Calculate the Eccentric Loading Effective Footing Area:

$A' := |B' \cdot L'|$ $\qquad A' = 90.0 \cdot ft^2 \qquad A' = 8.36\,m^{2.00}$

Calculations, Section 2: Bearing Capacity Coefficients **Checked By:**

Calculate Reduced Shear Strength Parameters if Local or Punching Shear is Possible:

$c := \begin{cases} 0.67 \cdot c & \text{if } F_{ps} = \text{"Y"} \\ c & \text{otherwise} \end{cases}$ $\qquad c = 0.0 \cdot psf \qquad c = 0.0 \cdot kPa$

$\phi := \begin{cases} \text{atan}(0.67 \cdot \tan(\phi)) & \text{if } F_{ps} = \text{"Y"} \\ \phi & \text{otherwise} \end{cases}$ $\qquad \phi = 36 \cdot deg$

Calculate Bearing Capacity Factors:

$N_q := \exp(\pi \cdot \tan(\phi)) \cdot \tan\left(45deg + \dfrac{\phi}{2}\right)^2$ $\qquad N_q = 37.752$

$N_c := \max\left[\left(N_q - 1\right) \cdot \cot(\max(\phi, 0.01deg)), 5.14\right]$ $\qquad \phi = 0.628 \qquad N_c = 50.585$

$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan(\phi)$ $\qquad N_\gamma = 56.311$

Calculate the Ground Water Factors Cwy and Cwq:

$C_{wq} := \begin{cases} 0.5 & \text{if } D_w = 0 \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w}{1.5 \cdot B + D_f} & \text{otherwise} \end{cases}$ $\qquad C_{wq} = 0.5$

$$C_{w\gamma} := \begin{cases} 0.5 & \text{if } D_w \le D_f \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w - D_f}{1.5 \cdot B} & \text{otherwise} \end{cases}$$

$$C_{w\gamma} = 0.5$$

Calculate Depth Factors:

$$\phi = 36 \cdot deg$$

$$\min\left(\frac{D_f}{B}, 8\right) = 0$$

$$dq_{42} := \begin{pmatrix} 0 & 1 \\ 1 & 1.15 \\ 2 & 1.20 \\ 4 & 1.25 \\ 8 & 1.30 \end{pmatrix} \qquad dq_{37} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.25 \\ 4 & 1.30 \\ 8 & 1.35 \end{pmatrix} \qquad dq_{32} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.30 \\ 4 & 1.35 \\ 8 & 1.40 \end{pmatrix}$$

The first columns of vectors above is Df/B. Correlation only valid for friction angles of 32 to 42 degrees; above 42 degrees,value for 42 degrees is considered conservative.

$$d_q := \begin{cases} \text{linterp}\left(dq_{42}^{\langle 0 \rangle}, dq_{42}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } \phi \ge 42deg \\ \text{linterp}\left(dq_{37}^{\langle 0 \rangle}, dq_{37}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 42deg > \phi \ge 37deg \\ \text{linterp}\left(dq_{32}^{\langle 0 \rangle}, dq_{32}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 37deg > \phi \ge 32deg \\ 1 & \text{otherwise} \end{cases}$$

$$d_q = 1$$

Calculate Footing Shape Factors:

$$s_c := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \left(\dfrac{N_q}{N_c}\right) & \text{if } \phi > 0 \\ 1 + \dfrac{B'}{5 \cdot L'} & \text{otherwise} \end{cases}$$

(all terms to go 1.0 for strip footing)

$$s_c = 1.207$$

$$s_q := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \tan(\phi) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_q = 1.202$$

$$s_\gamma := \begin{cases} 1 - 0.4 \cdot \left(\dfrac{B'}{L'}\right) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_\gamma = 0.889$$

Calculate Inclined Loading Factors:

$$n := \left(\frac{2 + \dfrac{L'}{B'}}{1 + \dfrac{L'}{B'}}\right) \cdot \cos(\theta)^2 + \left(\frac{2 + \dfrac{B'}{L'}}{1 + \dfrac{B'}{L'}}\right) \cdot \sin(\theta)^2$$

$$n = 1.217$$

$$i_q := \left(1 - \frac{H}{V + c \cdot B' \cdot L' \cdot \cot(\phi)}\right)^n$$

$$i_q = 1$$

$$i_c := \begin{cases} i_q - \left(\dfrac{1 - i_q}{N_q - 1}\right) & \text{if } \phi > 0\text{deg} \\ 1 - \left(\dfrac{n \cdot H}{c \cdot B' \cdot L' \cdot N_c}\right) & \text{otherwise} \end{cases}$$ $i_c = 1$

$$i_\gamma := \left(1 - \frac{H}{V + B' \cdot L' \cdot c \cdot \cot(\phi)}\right)^{n+1}$$ $i_\gamma = 1$

Calculate Modified Bearing Capacity Coefficients: $j = 0 \cdot \text{deg}$

$$N_{cm} := \begin{cases} N_c \cdot s_c \cdot i_c & \text{if } j = 0\text{deg} \\ N_{cslope} \cdot s_c \cdot i_c & \text{otherwise} \end{cases}$$ $N_{cm} = 61.072$

$$N_{qm} := \begin{cases} N_q \cdot s_q \cdot d_q \cdot i_q & \text{if } j = 0\text{deg} \\ 0 & \text{otherwise} \end{cases}$$ $N_{qm} = 45.372$

$$N_{\gamma m} := \begin{cases} N_\gamma \cdot s_\gamma \cdot i_\gamma & \text{if } j = 0\text{deg} \\ N_{\gamma slope} \cdot s_\gamma \cdot i_\gamma & \text{otherwise} \end{cases}$$ $N_{\gamma m} = 50.054$

Calculations, Section 3: Sliding Check **Checked By:**

Calculate the Maximum Resistance Force Between Footing and Foundation Soil for Sliding Failure:

$$P_{max} := V \cdot \tan(\delta) + B \cdot L \cdot c_a$$ $P_{max} = 185.4 \cdot \text{kip}$ $P_{max} = 824.6 \cdot \text{kN}$

Calculate the Factored Resistance Against Sliding Failure:

$$P_{fres} := P_{max} \cdot \phi_\tau$$ $P_{fres} = 148.308 \cdot \text{kip}$ $P_{fres} = 659.706 \cdot \text{kN}$

Check Sliding Factor of Safety:

$$Check_1 := \begin{cases} 1 & \text{if } H < P_{fres} \\ 0 & \text{otherwise} \end{cases}$$ $Check_1 = 1$

If $Check_1 = 0$, sliding factor of safety below acceptable value.

Calculations, Section 4: Bearing Capacity **Checked By:**

Calculate Ultimate Bearing Capacity: Eq. 10.6.3.1.2a-1 Note that g term is included in unit weight

$$q_n := c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$ $q_n = 8.0 \cdot \text{ksf}$ $q_n = 381.4 \cdot \text{kPa}$

Calculate Unfactored Bearing Capacity:

$$q_R := q_n \cdot \phi_b$$ $q_R = 3.6 \cdot \text{ksf}$ $q_R = 171.6 \cdot \text{kPa}$

Bearing Pressure: $q_L := \dfrac{V}{A'}$ $Check_2 := \begin{cases} 1 & \text{if } q_L < q_n \\ 0 & \text{otherwise} \end{cases}$ $q_L = 3.747 \cdot \text{ksf}$ $q_L = 179.4 \cdot \text{kPa}$ $Check_2 = 1$

Nominal (ultimate) bearing capacity:		**Ultimate slding resistance**		**Sliding OK (1) or not OK (0)?**
$q_n = 8 \cdot \text{ksf}$	$q_n = 381.411 \cdot \text{kPa}$	$P_{max} = 185 \cdot \text{kip}$	$P_{max} = 825 \cdot \text{kN}$	$Check_1 = 1$
Strength I factored bearing capacity		**Factored Sliding Resistance**		**Ultimate Bearing OK (1) or not OK (0)?**
$q_R = 3.6 \cdot \text{ksf}$	$q_R = 171.635 \cdot \text{kPa}$	$P_{fres} = 148 \cdot \text{kip}$	$P_{fres} = 660 \cdot \text{kN}$	$Check_2 = 1$

CALCULATION OF LRFD 8TH EDITION (2017) BEARING CAPACITY

Location: Pier Footings on Native Ground
Foundation: 10 feet Wide Footing footing

References

1. AASHTO, 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, American Association of State Highway and Transportation Officials.

Assumptions

1. Bearing capacity calculations account for foundation shape, possibility of local or punching shear, inclined load, eccentric loading, sloping ground, and ground water.
2. Calculations assume one, homogeneous soil unit. Two-layer soil systems not supported.

Unit Conversions **Checked By:**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $g = 32.174 \cdot \frac{ft}{s^2}$ $ksf := \frac{kip}{ft^2}$ $kPa := 1000Pa$ $kN := 1000N$ $kJ := 1000J$

Input Data **Checked By:**

Soil Cohesion:	$c := 0psf$	$c = 0.0 \cdot kPa$
Soil Friction Angle:	$\phi := 36deg$	
Total Soil Unit Weight:	$\gamma := 20\frac{kN}{m^3}$	$\gamma = 127.3 \cdot pcf$
Depth of Foundation Base below Ground Surface:	$D_f := 0m$	$D_f = 0.00 \cdot ft$
Foundation Width B (For Circular Footings B = L):	$B := 3.048m$	$B = 10.00 \cdot ft$
Foundation Length L:	$L := 5.4864m$	$L = 18.00 \cdot ft$
Depth of Ground Water from Ground Surface:	$D_w := 0ft$	$D_w = 0.00$
Slope of Adjacent Ground (if j>0, the modified Nγ and Nc apply below, Nq=0):	$j := 0deg$	
Calculate estimate reduction factor from Table 10.6.3.1.2c-1 or -2 and calculate teh reduced bearing capacity factors	$N_{\gamma slope} := 19$ $N_{cslope} := 0$	for β = 20.6 deg.
Is Local or Punching Shear Possible (Yes = "Y" and No = "N")?	$F_{ps} := "N"$	
Unfactored Vertical Load on Footing (Vertical):	$V := 1500kN$	$V = 337.2 \cdot kip$
Unfactored Horiz Load on Footing (Enter 0 for vertical load only):	$H := 0kip$	$H = 0.0 \cdot kN$
Orientation of Horizontal Load (Enter 0 for parallel to long axis L):	$\theta := 0deg$	
Moment in x-Dimension (Footing Width):	$M_x := 0kip \cdot ft$	$M_x = 0.0 \cdot kJ$
Moment in y-Dimension (Footing Length):	$M_y := 0kip \cdot ft$	$M_y = 0.0 \cdot kJ$
Adhesion Between Footing and Foundation Soil for Sliding:	$c_a := 0psf$	$c_a = 0.0 \cdot kPa$
Angle of Friction Between Footing and Foundation Soil for Sliding:	$\delta := 0.8 \cdot \phi$	$\delta = 28.8 \cdot deg$

Sliding Resistance Factor for the Strength Limit State: $\phi_T := 0.80$ CIP on sand

Bearing Resistance Factor for the Strenght Limit State: $\phi_b := 0.45$
Bearing Resistance Factor for Extreme State(scour, EQ, ice, impacts = 1.0)
Bearing Resistance Factor for Service State (Settlements and Servicability = 1.0)

This is a the Munfakh (2001) approach, ϕb varies from 0.45 to 0.5

An exception for service limit state 1 is that overall stability shall use resistance factors in Article 11.6.2.3

Calculations, Section 1: Bearing Pressures, Eccentricity Reduction **Checked By:**

Calculate Eccentricity in Footing "B" Direction:

$$e_B := \frac{M_y}{V} \qquad e_B = 0.0 \cdot ft \qquad e_B = 0.00$$

Calculate Eccentricity in Footing "L" Direction:

$$e_L := \frac{M_x}{V} \qquad e_L = 0.0 \cdot ft \qquad e_L = 0.00$$

Calculate Eccentric Loading Reduced Footing Dimensions:

$$B' := B - 2 \cdot e_B \qquad B' = 10.0 \cdot ft \qquad B' = 3.05\,m$$

$$L' := L - 2 \cdot e_L \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$$

Determine Effective Footing Dimensions based on any Eccentricity:

$$B' := \begin{cases} B' & \text{if } e_B > 0ft \\ B & \text{otherwise} \end{cases} \qquad B' = 10.0 \cdot ft \qquad B' = 3.05\,m$$

$$L' := \begin{cases} L' & \text{if } e_L > 0ft \\ L' & \text{otherwise} \end{cases} \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$$

Calculate the Eccentric Loading Effective Footing Area:

$$A' := |B' \cdot L'| \qquad A' = 180.0 \cdot ft^2 \qquad A' = 16.72\,m^{2.00}$$

Calculations, Section 2: Bearing Capacity Coefficients **Checked By:**

Calculate Reduced Shear Strength Parameters if Local or Punching Shear is Possible:

$$c := \begin{cases} 0.67 \cdot c & \text{if } F_{ps} = \text{"Y"} \\ c & \text{otherwise} \end{cases} \qquad c = 0.0 \cdot psf \qquad c = 0.0 \cdot kPa$$

$$\phi := \begin{cases} \operatorname{atan}(0.67 \cdot \tan(\phi)) & \text{if } F_{ps} = \text{"Y"} \\ \phi & \text{otherwise} \end{cases} \qquad \phi = 36 \cdot deg$$

Calculate Bearing Capacity Factors:

$$N_q := \exp(\pi \cdot \tan(\phi)) \cdot \tan\left(45deg + \frac{\phi}{2}\right)^2 \qquad N_q = 37.752$$

$$N_c := \max\left[\left(N_q - 1\right) \cdot \cot(\max(\phi, 0.01deg)), 5.14\right] \qquad \phi = 0.628 \qquad N_c = 50.585$$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan(\phi) \qquad N_\gamma = 56.311$$

Calculate the Ground Water Factors Cwy and Cwq:

$$C_{wq} := \begin{cases} 0.5 & \text{if } D_w = 0 \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \frac{D_w}{1.5 \cdot B + D_f} & \text{otherwise} \end{cases} \qquad C_{wq} = 0.5$$

$$C_{w\gamma} := \begin{cases} 0.5 & \text{if } D_w \le D_f \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w - D_f}{1.5 \cdot B} & \text{otherwise} \end{cases}$$

$C_{w\gamma} = 0.5$

Calculate Depth Factors:

$\phi = 36 \cdot deg$

$\min\left(\dfrac{D_f}{B}, 8\right) = 0$

$$dq_{42} := \begin{pmatrix} 0 & 1 \\ 1 & 1.15 \\ 2 & 1.20 \\ 4 & 1.25 \\ 8 & 1.30 \end{pmatrix} \qquad dq_{37} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.25 \\ 4 & 1.30 \\ 8 & 1.35 \end{pmatrix} \qquad dq_{32} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.30 \\ 4 & 1.35 \\ 8 & 1.40 \end{pmatrix}$$

The first columns of vectors above is Df/B. Correlation only valid for friction angles of 32 to 42 degrees; above 42 degrees,value for 42 degrees is considered conservative.

$$d_q := \begin{cases} \text{linterp}\left(dq_{42}^{\langle 0 \rangle}, dq_{42}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } \phi \ge 42deg \\ \text{linterp}\left(dq_{37}^{\langle 0 \rangle}, dq_{37}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 42deg > \phi \ge 37deg \\ \text{linterp}\left(dq_{32}^{\langle 0 \rangle}, dq_{32}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 37deg > \phi \ge 32deg \\ 1 & \text{otherwise} \end{cases}$$

$d_q = 1$

Calculate Footing Shape Factors:

$$s_c := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \left(\dfrac{N_q}{N_c}\right) & \text{if } \phi > 0 \\ 1 + \dfrac{B'}{5 \cdot L'} & \text{otherwise} \end{cases}$$

(all terms to go 1.0 for strip footing)

$s_c = 1.415$

$$s_q := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \tan(\phi) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$s_q = 1.404$

$$s_\gamma := \begin{cases} 1 - 0.4 \cdot \left(\dfrac{B'}{L'}\right) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$s_\gamma = 0.778$

Calculate Inclined Loading Factors:

$$n := \left(\frac{2 + \dfrac{L'}{B'}}{1 + \dfrac{L'}{B'}}\right) \cdot \cos(\theta)^2 + \left(\frac{2 + \dfrac{B'}{L'}}{1 + \dfrac{B'}{L'}}\right) \cdot \sin(\theta)^2$$

$n = 1.357$

$$i_q := \left(1 - \frac{H}{V + c \cdot B' \cdot L' \cdot \cot(\phi)}\right)^n$$

$i_q = 1$

$$i_c := \begin{cases} i_q - \left(\dfrac{1 - i_q}{N_q - 1}\right) & \text{if } \phi > 0\text{deg} \\ 1 - \left(\dfrac{n \cdot H}{c \cdot B' \cdot L' \cdot N_c}\right) & \text{otherwise} \end{cases}$$

$i_c = 1$

$$i_\gamma := \left(1 - \frac{H}{V + B' \cdot L' \cdot c \cdot \cot(\phi)}\right)^{n+1}$$

$i_\gamma = 1$

Calculate Modified Bearing Capacity Coefficients:

$j = 0 \cdot \text{deg}$

$$N_{cm} := \begin{cases} N_c \cdot s_c \cdot i_c & \text{if } j = 0\text{deg} \\ N_{cslope} \cdot s_c \cdot i_c & \text{otherwise} \end{cases}$$

$N_{cm} = 71.559$

$$N_{qm} := \begin{cases} N_q \cdot s_q \cdot d_q \cdot i_q & \text{if } j = 0\text{deg} \\ 0 & \text{otherwise} \end{cases}$$

$N_{qm} = 52.991$

$$N_{\gamma m} := \begin{cases} N_\gamma \cdot s_\gamma \cdot i_\gamma & \text{if } j = 0\text{deg} \\ N_{\gamma slope} \cdot s_\gamma \cdot i_\gamma & \text{otherwise} \end{cases}$$

$N_{\gamma m} = 43.797$

Calculations, Section 3: Sliding Check **Checked By:**

Calculate the Maximum Resistance Force Between Footing and Foundation Soil for Sliding Failure:

$P_{max} := V \cdot \tan(\delta) + B \cdot L \cdot c_a$

$P_{max} = 185.4 \cdot \text{kip}$ $P_{max} = 824.6 \cdot \text{kN}$

Calculate the Factored Resistance Against Sliding Failure:

$P_{fres} := P_{max} \cdot \phi_\tau$

$P_{fres} = 148.308 \cdot \text{kip}$ $P_{fres} = 659.706 \cdot \text{kN}$

Check Sliding Factor of Safety:

$$Check_1 := \begin{cases} 1 & \text{if } H < P_{fres} \\ 0 & \text{otherwise} \end{cases}$$

$Check_1 = 1$

If $Check_1 = 0$, sliding factor of safety below acceptable value.

Calculations, Section 4: Bearing Capacity **Checked By:**

Calculate Ultimate Bearing Capacity: Eq. 10.6.3.1.2a-1 Note that g term is included in unit weight

$q_n := c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$

$q_n = 13.9 \cdot \text{ksf}$ $q_n = 667.5 \cdot \text{kPa}$

Calculate Unfactored Bearing Capacity:

$q_R := q_n \cdot \phi_b$

$q_R = 6.3 \cdot \text{ksf}$ $q_R = 300.4 \cdot \text{kPa}$

Bearing Pressure: $q_L := \dfrac{V}{A'}$

$$Check_2 := \begin{cases} 1 & \text{if } q_L < q_n \\ 0 & \text{otherwise} \end{cases}$$

$q_L = 1.873 \cdot \text{ksf}$ $q_L = 89.7 \cdot \text{kPa}$

$Check_2 = 1$

Nominal (ultimate) bearing capacity:

$q_n = 13.9 \cdot \text{ksf}$ $q_n = 667.469 \cdot \text{kPa}$

Ultimate slding resistance

$P_{max} = 185 \cdot \text{kip}$ $P_{max} = 825 \cdot \text{kN}$

Sliding OK (1) or not OK (0)?

$Check_1 = 1$

Strength I factored bearing capacity

$q_R = 6.3 \cdot \text{ksf}$ $q_R = 300.361 \cdot \text{kPa}$

Factored Sliding Resistance

$P_{fres} = 148 \cdot \text{kip}$ $P_{fres} = 660 \cdot \text{kN}$

Ultimate Bearing OK (1) or not OK (0)?

$Check_2 = 1$

CORESTONE ENGINEERING, INC.		**Date:**	6/27/2019
1345 Capital Blvd, Suite B, Reno, NV 89502		**Revision No:**	2019 March - 1
		Developed by:	JWP/PV
Project Name:	**San Bernardino Class 1 Bike Trail Project**	**Calculated by:**	PV
Project Number:	**5012-02-1** B-02 Data	**Checked by:**	pv
Design Case:	**Pathway Bridge - Piers (B=5 ft)**		

SETTLEMENT USING AASHTO-MODIFIED "HOUGH METHOD"

Only cells with blue background and blue text should be modified

AASHTO, 2007, LRFD Design Manual 4th Edition p 10-55 Same for AASHTO 2017

Hough, 1959, Compressibility as the Basis for Soil Bearing Value, Journal of the Soil Mechanics and Foundations Division, ASCE SM4, August 1959

Foundation Load	1900	kN	427.1	kips		
Foundation Depth	0.9144	m	3.0	ft		
Foundation Width	1.524	**m**	5	ft	**4746.0 psf**	
Foundation Length	5.4864	**m**	18	ft	For 1 inch settlement (Service Value)	
Depth of Influence (3B)	5.4864	m	18.0	ft	check	1.00 inch
Depth to Water Table	0	m				

Depth m	Unit Weight kN/m^3	Total Stress kPa	Eff Stress kPa	Inc Stress kPa	Hough C'	Settlement m	depth ft
0.000	20.0	0	0	NA	NA	NA	0
0.762	20.0	15	8	0.0	200	0.00000	2.5
1.524	20.0	30	16	179.4	100	0.00837	5
2.286	20.0	46	23	139.3	150	0.00429	7.5
3.048	20.0	61	31	111.9	175	0.00289	10
3.810	20.0	76	39	92.2	100	0.00403	12.5
4.572	20.0	91	47	77.5	75	0.00432	15
5.334	20.0	107	54	66.1	175	0.00151	17.5
6.096	20.0	122	62	57.2	175	0.00000	20
6.858	20.0	137	70	50.0	175	0.00000	22.5
7.620	20.0	152	78	44.1	175	0.00000	25
8.382	20.0	168	85	39.2	175	0.00000	27.5
9.144	20.0	183	93	35.1	175	0.00000	30
9.906	20.0	198	101	31.6	175	0.00000	32.5
10.668	20.0	213	109	28.6	175	0.00000	35
11.430	20.0	229	116	26.1	175	0.00000	37.5
12.192	20.0	244	124	23.8	175	0.00000	40
12.954	20.0	259	132	21.9	175	0.00000	42.5
13.716	20.0	274	140	20.2	175	0.00000	45
14.478	20.0	290	148	18.6	200	0.00000	47.5
15.240	20.0	305	155	17.3	200	0.00000	50
16.002	20.0	320	163	16.1	200	0.00000	52.5
16.764	20.0	335	171	15.0	200	0.00000	55
17.526	20.0	351	179	14.0	200	0.00000	57.5
18.288	20.0	366	186	13.1	200	0.00000	60
19.050	20.0	381	194	12.3	200	0.00000	62.5
19.812	20.0	396	202	11.6	200	0.00000	65
20.574	20.0	411	210	10.9	200	0.00000	67.5
21.336	20.0	427	217	10.3	200	0.00000	70
22.098	20.0	442	225	9.8	200	0.00000	72.5
22.860	20.0	457	233	9.2	200	0.00000	75
23.622	20.0	472	241	8.8	200	0.00000	77.5
24.384	20.0	488	248	8.3	200	0.00000	80

0.91	Df, m	1.5	B, m	0.0254 m 25 mm
		5.5	L, m	
				1.00 inches
		227.2	q, kN/m^2	

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1345 Capital Blvd, Suite B, Reno, NV 89502

Date: 6/27/2019
Revision No: 2019 March - 1
Developed by: JWP/PV
Calculated by: PV
Checked by: pv

Project Name: **San Bernardino Class 1 Bike Trail Project**
Project Number: **5012-02-1** B-02 Data
Design Case: **Pathway Bridge - Piers (B=10 ft)**

SETTLEMENT USING AASHTO-MODIFIED "HOUGH METHOD"

Only cells with blue background and blue text should be modified

AASHTO, 2007, LRFD Design Manual 4th Edition p 10-55 Same for AASHTO 2017

Hough, 1959, Compressibility as the Basis for Soil Bearing Value, Journal of the Soil Mechanics and Foundations Division, ASCE SM4, August 1959

Foundation Load	2100 kN	472.1 kips			
Foundation Depth	0.9144 m	3.0 ft			
Foundation Width	3.048 **m**	10 ft	**2622.8 psf**		
Foundation Length	5.4864 **m**	18 ft	For 1 inch settlement (Service Value)		
Depth of Influence (3B)	10.0584 m	33.0 ft	check	0.99 inch	
Depth to Water Table	0 m				

Depth m	Unit Weight kN/m^3	Total Stress kPa	Eff Stress kPa	Inc Stress kPa	Hough C'	Settlement m	depth ft
0.000	20.0	0	0	NA	NA	NA	0
0.762	20.0	15	8	0.0	200	0.00000	2.5
1.524	20.0	30	16	108.2	100	0.00687	5
2.286	20.0	46	23	91.1	150	0.00351	7.5
3.048	20.0	61	31	77.9	175	0.00237	10
3.810	20.0	76	39	67.4	100	0.00333	12.5
4.572	20.0	91	47	58.9	75	0.00360	15
5.334	20.0	107	54	51.9	175	0.00127	17.5
6.096	20.0	122	62	46.1	175	0.00105	20
6.858	20.0	137	70	41.2	175	0.00088	22.5
7.620	20.0	152	78	37.1	175	0.00074	25
8.382	20.0	168	85	33.6	175	0.00063	27.5
9.144	20.0	183	93	30.5	175	0.00054	30
9.906	20.0	198	101	27.9	175	0.00046	32.5
10.668	20.0	213	109	25.6	175	0.00000	35
11.430	20.0	229	116	23.5	175	0.00000	37.5
12.192	20.0	244	124	21.7	175	0.00000	40
12.954	20.0	259	132	20.1	175	0.00000	42.5
13.716	20.0	274	140	18.7	175	0.00000	45
14.478	20.0	290	148	17.4	200	0.00000	47.5
15.240	20.0	305	155	16.3	200	0.00000	50
16.002	20.0	320	163	15.2	200	0.00000	52.5
16.764	20.0	335	171	14.3	200	0.00000	55
17.526	20.0	351	179	13.4	200	0.00000	57.5
18.288	20.0	366	186	12.6	200	0.00000	60
19.050	20.0	381	194	11.9	200	0.00000	62.5
19.812	20.0	396	202	11.3	200	0.00000	65
20.574	20.0	411	210	10.6	200	0.00000	67.5
21.336	20.0	427	217	10.1	200	0.00000	70
22.098	20.0	442	225	9.6	200	0.00000	72.5
22.860	20.0	457	233	9.1	200	0.00000	75
23.622	20.0	472	241	8.7	200	0.00000	77.5
24.384	20.0	488	248	8.2	200	0.00000	80

0.91 Df, m

3.0 B, m

5.5 L, m

125.6 q, kN/m^2

0.0252 m
25 mm

0.99 inches

CORESTONE ENGINEERING, INC.
1345 Capital Blvd, Suite B, Reno, NV 89502; ph. (775) 636-5916

Date: 6/25/2019
Revision Number: 0319
Developed By: JWP/PV

Project Name: San Bernardino Class 1 Bike Trail Project
Project Number: 5012-02-1

Calculated By: PV
Checked By:

CALCULATION OF LRFD 8TH EDITION (2017) BEARING CAPACITY

Location: Abutment Footings on Embankment Fill 2H:1V Slope
Foundation: 5 feet Wide Footing footing

References

1. AASHTO, 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, American Association of State Highway and Transportation Officials.

Assumptions

1. Bearing capacity calculations account for foundation shape, possibility of local or punching shear, inclined load, eccentric loading, sloping ground, and ground water.
2. Calculations assume one, homogeneous soil unit. Two-layer soil systems not supported.

Unit Conversions

Checked By:

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $g = 32.174 \cdot \frac{ft}{s^2}$ $ksf := \frac{kip}{ft^2}$ $kPa := 1000Pa$ $kN := 1000N$ $kJ := 1000J$

Input Data

Checked By:

Soil Cohesion:	$c := 0psf$	$c = 0.0 \cdot kPa$
Soil Friction Angle:	$\phi := 36deg$	
Total Soil Unit Weight:	$\gamma := 20\frac{kN}{m^3}$	$\gamma = 127.3 \cdot pcf$
Depth of Foundation Base below Ground Surface:	$D_f := 0m$	$D_f = 0.00 \cdot ft$
Foundation Width B (For Circular Footings B = L):	$B := 1.524m$	$B = 5.00 \cdot ft$
Foundation Length L:	$L := 5.4864m$	$L = 18.00 \cdot ft$
Depth of Ground Water from Ground Surface:	$D_w := 6ft$	$D_w = 1.83\,m$
Slope of Adjacent Ground (if j>0, the modified Nγ and Nc apply below, Nq=0):	$j := 26.56deg$	
Calculate estimate reduction factor from Table 10.6.3.1.2c-1 or -2 and calculate teh reduced bearing capacity factors	$N_{\gamma slope} := 15.25$ for β = 20.6 deg. $N_{cslope} := 0$	
Is Local or Punching Shear Possible (Yes = "Y" and No = "N")?	$F_{ps} :=$ "N"	
Unfactored Vertical Load on Footing (Vertical):	$V := 1500kN$	$V = 337.2 \cdot kip$
Unfactored Horiz Load on Footing (Enter 0 for vertical load only):	$H := 0kip$	$H = 0.0 \cdot kN$
Orientation of Horizontal Load (Enter 0 for parallel to long axis L):	$\theta := 0deg$	
Moment in x-Dimension (Footing Width):	$M_x := 0kip \cdot ft$	$M_x = 0.0 \cdot kJ$
Moment in y-Dimension (Footing Length):	$M_y := 0kip \cdot ft$	$M_y = 0.0 \cdot kJ$
Adhesion Between Footing and Foundation Soil for Sliding:	$c_a := 0psf$	$c_a = 0.0 \cdot kPa$
Angle of Friction Between Footing and Foundation Soil for Sliding:	$\delta := 0.8 \cdot \phi$	$\delta = 28.8 \cdot deg$

Sliding Resistance Factor for the Strength Limit State: $\phi_\tau := 0.80$ CIP on sand

Bearing Resistance Factor for the Strenght Limit State: $\phi_b := 0.45$ This is a the Munfakh (2001) approach, ϕb varies from 0.45 to 0.5
Bearing Resistance Factor for Extreme State(scour, EQ, ice, impacts = 1.0)
Bearing Resistance Factor for Service State (Settlements and Servicability = 1.0)

An exception for service limit state 1 is that overall stability shall use resistance factors in Article 11.6.2.3

Calculations, Section 1: Bearing Pressures, Eccentricity Reduction **Checked By:**

Calculate Eccentricity in Footing "B" Direction:

$$e_B := \frac{M_y}{V} \qquad e_B = 0.0 \cdot ft \qquad e_B = 0.00$$

Calculate Eccentricity in Footing "L" Direction:

$$e_L := \frac{M_x}{V} \qquad e_L = 0.0 \cdot ft \qquad e_L = 0.00$$

Calculate Eccentric Loading Reduced Footing Dimensions:

$$B' := B - 2 \cdot e_B \qquad B' = 5.0 \cdot ft \qquad B' = 1.52\,m$$

$$L' := L - 2 \cdot e_L \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$$

Determine Effective Footing Dimensions based on any Eccentricity:

$$B' := \begin{cases} B' & \text{if } e_B > 0ft \\ B & \text{otherwise} \end{cases} \qquad B' = 5.0 \cdot ft \qquad B' = 1.52\,m$$

$$L' := \begin{cases} L' & \text{if } e_L > 0ft \\ L' & \text{otherwise} \end{cases} \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\,m$$

Calculate the Eccentric Loading Effective Footing Area:

$$A' := |B' \cdot L'| \qquad A' = 90.0 \cdot ft^2 \qquad A' = 8.36\,m^{2.00}$$

Calculations, Section 2: Bearing Capacity Coefficients **Checked By:**

Calculate Reduced Shear Strength Parameters if Local or Punching Shear is Possible:

$$c := \begin{cases} 0.67 \cdot c & \text{if } F_{ps} = \text{"Y"} \\ c & \text{otherwise} \end{cases} \qquad c = 0.0 \cdot psf \qquad c = 0.0 \cdot kPa$$

$$\phi := \begin{cases} \operatorname{atan}(0.67 \cdot \tan(\phi)) & \text{if } F_{ps} = \text{"Y"} \\ \phi & \text{otherwise} \end{cases} \qquad \phi = 36 \cdot deg$$

Calculate Bearing Capacity Factors:

$$N_q := \exp(\pi \cdot \tan(\phi)) \cdot \tan\left(45deg + \frac{\phi}{2}\right)^2 \qquad N_q = 37.752$$

$$N_c := \max\left[\left(N_q - 1\right) \cdot \cot(\max(\phi, 0.01deg)), 5.14\right] \qquad \phi = 0.628 \qquad N_c = 50.585$$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan(\phi) \qquad N_\gamma = 56.311$$

Calculate the Ground Water Factors Cwy and Cwq:

$$C_{wq} := \begin{cases} 0.5 & \text{if } D_w = 0 \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w}{1.5 \cdot B + D_f} & \text{otherwise} \end{cases} \qquad C_{wq} = 0.9$$

$$C_{w\gamma} := \begin{cases} 0.5 & \text{if } D_w \le D_f \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w - D_f}{1.5 \cdot B} & \text{otherwise} \end{cases}$$

$$C_{w\gamma} = 0.9$$

Calculate Depth Factors:

$$\phi = 36 \cdot \text{deg}$$

$$\min\left(\frac{D_f}{B}, 8\right) = 0$$

$$dq_{42} := \begin{pmatrix} 0 & 1 \\ 1 & 1.15 \\ 2 & 1.20 \\ 4 & 1.25 \\ 8 & 1.30 \end{pmatrix} \qquad dq_{37} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.25 \\ 4 & 1.30 \\ 8 & 1.35 \end{pmatrix} \qquad dq_{32} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.30 \\ 4 & 1.35 \\ 8 & 1.40 \end{pmatrix}$$

The first columns of vectors above is Df/B. Correlation only valid for friction angles of 32 to 42 degrees; above 42 degrees,value for 42 degrees is considered conservative.

$$d_q := \begin{cases} \text{linterp}\left(dq_{42}^{\langle 0 \rangle}, dq_{42}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } \phi \ge 42\text{deg} \\ \text{linterp}\left(dq_{37}^{\langle 0 \rangle}, dq_{37}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 42\text{deg} > \phi \ge 37\text{deg} \\ \text{linterp}\left(dq_{32}^{\langle 0 \rangle}, dq_{32}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 37\text{deg} > \phi \ge 32\text{deg} \\ 1 & \text{otherwise} \end{cases}$$

$$d_q = 1$$

Calculate Footing Shape Factors:

$$s_c := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \left(\dfrac{N_q}{N_c}\right) & \text{if } \phi > 0 \\ 1 + \dfrac{B'}{5 \cdot L'} & \text{otherwise} \end{cases}$$

(all terms to go 1.0 for strip footing)

$$s_c = 1.207$$

$$s_q := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \tan(\phi) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_q = 1.202$$

$$s_\gamma := \begin{cases} 1 - 0.4 \cdot \left(\dfrac{B'}{L'}\right) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_\gamma = 0.889$$

Calculate Inclined Loading Factors:

$$n := \left(\frac{2 + \dfrac{L'}{B'}}{1 + \dfrac{L'}{B'}}\right) \cdot \cos(\theta)^2 + \left(\frac{2 + \dfrac{B'}{L'}}{1 + \dfrac{B'}{L'}}\right) \cdot \sin(\theta)^2$$

$$n = 1.217$$

$$i_q := \left(1 - \frac{H}{V + c \cdot B' \cdot L' \cdot \cot(\phi)}\right)^n$$

$$i_q = 1$$

$$i_c := \begin{cases} i_q - \left(\dfrac{1 - i_q}{N_q - 1}\right) & \text{if } \phi > 0\text{deg} \\ 1 - \left(\dfrac{n \cdot H}{c \cdot B' \cdot L' \cdot N_c}\right) & \text{otherwise} \end{cases}$$

$i_c = 1$

$$i_\gamma := \left(1 - \frac{H}{V + B' \cdot L' \cdot c \cdot \cot(\phi)}\right)^{n+1}$$

$i_\gamma = 1$

<u>Calculate Modified Bearing Capacity Coefficients:</u>

$j = 26.56 \cdot \text{deg}$

$$N_{cm} := \begin{cases} N_c \cdot s_c \cdot i_c & \text{if } j = 0\text{deg} \\ N_{cslope} \cdot s_c \cdot i_c & \text{otherwise} \end{cases}$$

$N_{cm} = 0$

$$N_{qm} := \begin{cases} N_q \cdot s_q \cdot d_q \cdot i_q & \text{if } j = 0\text{deg} \\ 0 & \text{otherwise} \end{cases}$$

$N_{qm} = 0$

$$N_{\gamma m} := \begin{cases} N_\gamma \cdot s_\gamma \cdot i_\gamma & \text{if } j = 0\text{deg} \\ N_{\gamma slope} \cdot s_\gamma \cdot i_\gamma & \text{otherwise} \end{cases}$$

$N_{\gamma m} = 13.556$

Calculations, Section 3: Sliding Check **Checked By:**

<u>Calculate the Maximum Resistance Force Between Footing and Foundation Soil for Sliding Failure:</u>

$P_{max} := V \cdot \tan(\delta) + B \cdot L \cdot c_a$

$P_{max} = 185.4 \cdot \text{kip}$ $P_{max} = 824.6 \cdot \text{kN}$

<u>Calculate the Factored Resistance Against Sliding Failure:</u>

$P_{fres} := P_{max} \cdot \phi_\tau$

$P_{fres} = 148.308 \cdot \text{kip}$ $P_{fres} = 659.706 \cdot \text{kN}$

<u>Check Sliding Factor of Safety:</u>

$$\text{Check}_1 := \begin{cases} 1 & \text{if } H < P_{fres} \\ 0 & \text{otherwise} \end{cases}$$

$\text{Check}_1 = 1$

If $\text{Check}_1 = 0$, sliding factor of safety below acceptable value.

Calculations, Section 4: Bearing Capacity **Checked By:**

<u>Calculate Ultimate Bearing Capacity: Eq. 10.6.3.1.2a-1 Note that g term is included in unit weight</u>

$q_n := c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$

$q_n = 3.9 \cdot \text{ksf}$ $q_n = 185.9 \cdot \text{kPa}$

<u>Calculate Unfactored Bearing Capacity:</u>

$q_R := q_n \cdot \phi_b$

$q_R = 1.7 \cdot \text{ksf}$ $q_R = 83.7 \cdot \text{kPa}$

Bearing Pressure: $q_L := \dfrac{V}{A'}$

$$\text{Check}_2 := \begin{cases} 1 & \text{if } q_L < q_n \\ 0 & \text{otherwise} \end{cases}$$

$q_L = 3.747 \cdot \text{ksf}$ $q_L = 179.4 \cdot \text{kPa}$

$\text{Check}_2 = 1$

Nominal (ultimate) bearing capacity:

$q_n = 3.9 \cdot \text{ksf}$ $q_n = 185.928 \cdot \text{kPa}$

Ultimate slding resistance

$P_{max} = 185 \cdot \text{kip}$ $P_{max} = 825 \cdot \text{kN}$

Sliding OK (1) or not OK (0)?

$\text{Check}_1 = 1$

Strength I factored bearing capacity

$q_R = 1.7 \cdot \text{ksf}$ $q_R = 83.668 \cdot \text{kPa}$

Factored Sliding Resistance

$P_{fres} = 148 \cdot \text{kip}$ $P_{fres} = 660 \cdot \text{kN}$

Ultimate Bearing OK (1) or not OK (0)?

$\text{Check}_2 = 1$

CORESTONE ENGINEERING, INC.
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Date: 6/25/2019
Revision Number: 0319
Developed By: JWP/PV

Project Name: San Bernardino Class 1 Bike Trail Project
Project Number: 5012-02-1

Calculated By: PV
Checked By:

CALCULATION OF LRFD 8TH EDITION (2017) BEARING CAPACITY

Location: Abutment Footings on Embankment Fill 2H:1V Slope
Foundation: 10 feet Wide Footing footing

References

1. AASHTO, 2017, AASHTO LRFD Bridge Design Specifications, 8th Edition, American Association of State Highway and Transportation Officials.

Assumptions

1. Bearing capacity calculations account for foundation shape, possibility of local or punching shear, inclined load, eccentric loading, sloping ground, and ground water.
2. Calculations assume one, homogeneous soil unit. Two-layer soil systems not supported.

Unit Conversions **Checked By:**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $kip := 1000lbf$ $g = 32.174 \cdot \frac{ft}{s^2}$ $ksf := \frac{kip}{ft^2}$ $kPa := 1000Pa$ $kN := 1000N$ $kJ := 1000J$

Input Data **Checked By:**

Soil Cohesion:	$c := 0psf$	$c = 0.0 \cdot kPa$
Soil Friction Angle:	$\phi := 36deg$	
Total Soil Unit Weight:	$\gamma := 20 \frac{kN}{m^3}$	$\gamma = 127.3 \cdot pcf$
Depth of Foundation Base below Ground Surface:	$D_f := 0m$	$D_f = 0.00 \cdot ft$
Foundation Width B (For Circular Footings B = L):	$B := 3.048m$	$B = 10.00 \cdot ft$
Foundation Length L:	$L := 5.4864m$	$L = 18.00 \cdot ft$
Depth of Ground Water from Ground Surface:	$D_w := 6ft$	$D_w = 1.83\,m$
Slope of Adjacent Ground (if j>0, the modified Nγ and Nc apply below, Nq=0):	$j := 26.56deg$	
Calculate estimate reduction factor from Table 10.6.3.1.2c-1 or -2 and calculate teh reduced bearing capacity factors	$N_{\gamma slope} := 21.22$ $N_{cslope} := 0$	for β = 20.6 deg.
Is Local or Punching Shear Possible (Yes = "Y" and No = "N")?	$F_{ps} := "N"$	
Unfactored Vertical Load on Footing (Vertical):	$V := 1500kN$	$V = 337.2 \cdot kip$
Unfactored Horiz Load on Footing (Enter 0 for vertical load only):	$H := 0kip$	$H = 0.0 \cdot kN$
Orientation of Horizontal Load (Enter 0 for parallel to long axis L):	$\theta := 0deg$	
Moment in x-Dimension (Footing Width):	$M_x := 0kip \cdot ft$	$M_x = 0.0 \cdot kJ$
Moment in y-Dimension (Footing Length):	$M_y := 0kip \cdot ft$	$M_y = 0.0 \cdot kJ$
Adhesion Between Footing and Foundation Soil for Sliding:	$c_a := 0psf$	$c_a = 0.0 \cdot kPa$
Angle of Friction Between Footing and Foundation Soil for Sliding:	$\delta := 0.8 \cdot \phi$	$\delta = 28.8 \cdot deg$

Sliding Resistance Factor for the Strength Limit State: $\phi_T := 0.80$ CIP on sand

Bearing Resistance Factor for the Strenght Limit State: $\phi_b := 0.45$ This is a the Munfakh (2001) approach, ϕb varies from 0.45 to 0.5
Bearing Resistance Factor for Extreme State(scour, EQ, ice, impacts = 1.0)
Bearing Resistance Factor for Service State (Settlements and Servicability = 1.0)

An exception for service limit state 1 is that overall stability shall use resistance factors in Article 11.6.2.3

Calculations, Section 1: Bearing Pressures, Eccentricity Reduction **Checked By:**

Calculate Eccentricity in Footing "B" Direction:

$$e_B := \frac{M_y}{V} \qquad e_B = 0.0 \cdot ft \qquad e_B = 0.00$$

Calculate Eccentricity in Footing "L" Direction:

$$e_L := \frac{M_x}{V} \qquad e_L = 0.0 \cdot ft \qquad e_L = 0.00$$

Calculate Eccentric Loading Reduced Footing Dimensions:

$$B' := B - 2 \cdot e_B \qquad B' = 10.0 \cdot ft \qquad B' = 3.05\, m$$

$$L' := L - 2 \cdot e_L \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\, m$$

Determine Effective Footing Dimensions based on any Eccentricity:

$$B' := \begin{cases} B' & \text{if } e_B > 0ft \\ B & \text{otherwise} \end{cases} \qquad B' = 10.0 \cdot ft \qquad B' = 3.05\, m$$

$$L' := \begin{cases} L' & \text{if } e_L > 0ft \\ L' & \text{otherwise} \end{cases} \qquad L' = 18.0 \cdot ft \qquad L' = 5.49\, m$$

Calculate the Eccentric Loading Effective Footing Area:

$$A' := |B' \cdot L'| \qquad A' = 180.0 \cdot ft^2 \qquad A' = 16.72\, m^{2.00}$$

Calculations, Section 2: Bearing Capacity Coefficients **Checked By:**

Calculate Reduced Shear Strength Parameters if Local or Punching Shear is Possible:

$$c := \begin{cases} 0.67 \cdot c & \text{if } F_{ps} = \text{"Y"} \\ c & \text{otherwise} \end{cases} \qquad c = 0.0 \cdot psf \qquad c = 0.0 \cdot kPa$$

$$\phi := \begin{cases} \operatorname{atan}(0.67 \cdot \tan(\phi)) & \text{if } F_{ps} = \text{"Y"} \\ \phi & \text{otherwise} \end{cases} \qquad \phi = 36 \cdot deg$$

Calculate Bearing Capacity Factors:

$$N_q := \exp(\pi \cdot \tan(\phi)) \cdot \tan\left(45deg + \frac{\phi}{2}\right)^2 \qquad N_q = 37.752$$

$$N_c := \max\left[\left(N_q - 1\right) \cdot \cot(\max(\phi, 0.01deg)), 5.14\right] \qquad \phi = 0.628 \qquad N_c = 50.585$$

$$N_\gamma := 2 \cdot \left(N_q + 1\right) \cdot \tan(\phi) \qquad N_\gamma = 56.311$$

Calculate the Ground Water Factors Cwy and Cwq:

$$C_{wq} := \begin{cases} 0.5 & \text{if } D_w = 0 \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w}{1.5 \cdot B + D_f} & \text{otherwise} \end{cases} \qquad C_{wq} = 0.7$$

$$C_{w\gamma} := \begin{cases} 0.5 & \text{if } D_w \le D_f \\ 1 & \text{if } D_w > 1.5 \cdot B + D_f \\ 0.5 + 0.5 \cdot \dfrac{D_w - D_f}{1.5 \cdot B} & \text{otherwise} \end{cases}$$

$$C_{w\gamma} = 0.7$$

Calculate Depth Factors:

$$\phi = 36 \cdot \text{deg}$$

$$\min\left(\frac{D_f}{B}, 8\right) = 0$$

$$dq_{42} := \begin{pmatrix} 0 & 1 \\ 1 & 1.15 \\ 2 & 1.20 \\ 4 & 1.25 \\ 8 & 1.30 \end{pmatrix} \quad dq_{37} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.25 \\ 4 & 1.30 \\ 8 & 1.35 \end{pmatrix} \quad dq_{32} := \begin{pmatrix} 0 & 1 \\ 1 & 1.20 \\ 2 & 1.30 \\ 4 & 1.35 \\ 8 & 1.40 \end{pmatrix}$$

The first columns of vectors above is Df/B. Correlation only valid for friction angles of 32 to 42 degrees; above 42 degrees,value for 42 degrees is considered conservative.

$$d_q := \begin{cases} \text{linterp}\left(dq_{42}^{\langle 0 \rangle}, dq_{42}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } \phi \ge 42\text{deg} \\ \text{linterp}\left(dq_{37}^{\langle 0 \rangle}, dq_{37}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 42\text{deg} > \phi \ge 37\text{deg} \\ \text{linterp}\left(dq_{32}^{\langle 0 \rangle}, dq_{32}^{\langle 1 \rangle}, \min\left(\dfrac{D_f}{B}, 8\right)\right) & \text{if } 37\text{deg} > \phi \ge 32\text{deg} \\ 1 & \text{otherwise} \end{cases}$$

$$d_q = 1$$

Calculate Footing Shape Factors:

$$s_c := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \left(\dfrac{N_q}{N_c}\right) & \text{if } \phi > 0 \\ 1 + \dfrac{B'}{5 \cdot L'} & \text{otherwise} \end{cases}$$

(all terms to go 1.0 for strip footing)

$$s_c = 1.415$$

$$s_q := \begin{cases} 1 + \left(\dfrac{B'}{L'}\right) \cdot \tan(\phi) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_q = 1.404$$

$$s_\gamma := \begin{cases} 1 - 0.4 \cdot \left(\dfrac{B'}{L'}\right) & \text{if } \phi > 0 \\ 1 & \text{otherwise} \end{cases}$$

$$s_\gamma = 0.778$$

Calculate Inclined Loading Factors:

$$n := \left(\frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}}\right) \cdot \cos(\theta)^2 + \left(\frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}}\right) \cdot \sin(\theta)^2$$

$$n = 1.357$$

$$i_q := \left(1 - \frac{H}{V + c \cdot B' \cdot L' \cdot \cot(\phi)}\right)^n$$

$$i_q = 1$$

$$i_c := \begin{cases} i_q - \left(\dfrac{1 - i_q}{N_q - 1}\right) & \text{if } \phi > 0\text{deg} \\ 1 - \left(\dfrac{n \cdot H}{c \cdot B' \cdot L' \cdot N_c}\right) & \text{otherwise} \end{cases}$$

$i_c = 1$

$$i_\gamma := \left(1 - \frac{H}{V + B' \cdot L' \cdot c \cdot \cot(\phi)}\right)^{n+1}$$

$i_\gamma = 1$

Calculate Modified Bearing Capacity Coefficients:

$j = 26.56 \cdot \text{deg}$

$$N_{cm} := \begin{cases} N_c \cdot s_c \cdot i_c & \text{if } j = 0\text{deg} \\ N_{cslope} \cdot s_c \cdot i_c & \text{otherwise} \end{cases}$$

$N_{cm} = 0$

$$N_{qm} := \begin{cases} N_q \cdot s_q \cdot d_q \cdot i_q & \text{if } j = 0\text{deg} \\ 0 & \text{otherwise} \end{cases}$$

$N_{qm} = 0$

$$N_{\gamma m} := \begin{cases} N_\gamma \cdot s_\gamma \cdot i_\gamma & \text{if } j = 0\text{deg} \\ N_{\gamma slope} \cdot s_\gamma \cdot i_\gamma & \text{otherwise} \end{cases}$$

$N_{\gamma m} = 16.504$

Calculations, Section 3: Sliding Check **Checked By:**

Calculate the Maximum Resistance Force Between Footing and Foundation Soil for Sliding Failure:

$$P_{max} := V \cdot \tan(\delta) + B \cdot L \cdot c_a$$

$P_{max} = 185.4 \cdot \text{kip}$ $P_{max} = 824.6 \cdot \text{kN}$

Calculate the Factored Resistance Against Sliding Failure:

$$P_{fres} := P_{max} \cdot \phi_\tau$$

$P_{fres} = 148.308 \cdot \text{kip}$ $P_{fres} = 659.706 \cdot \text{kN}$

Check Sliding Factor of Safety:

$$Check_1 := \begin{cases} 1 & \text{if } H < P_{fres} \\ 0 & \text{otherwise} \end{cases}$$

$Check_1 = 1$

If $Check_1 = 0$, sliding factor of safety below acceptable value.

Calculations, Section 4: Bearing Capacity **Checked By:**

Calculate Ultimate Bearing Capacity: Eq. 10.6.3.1.2a-1 Note that g term is included in unit weight

$$q_n := c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B' \cdot N_{\gamma m} \cdot C_{w\gamma}$$

$q_n = 7.4 \cdot \text{ksf}$ $q_n = 352.1 \cdot \text{kPa}$

Calculate Unfactored Bearing Capacity:

$$q_R := q_n \cdot \phi_b$$

$q_R = 3.3 \cdot \text{ksf}$ $q_R = 158.5 \cdot \text{kPa}$

Bearing Pressure: $q_L := \dfrac{V}{A'}$

$q_L = 1.873 \cdot \text{ksf}$ $q_L = 89.7 \cdot \text{kPa}$

$$Check_2 := \begin{cases} 1 & \text{if } q_L < q_n \\ 0 & \text{otherwise} \end{cases}$$

$Check_2 = 1$

Nominal (ultimate) bearing capacity:		Ultimate slding resistance		Sliding OK (1) or not OK (0)?
$q_n = 7.4 \cdot \text{ksf}$	$q_n = 352.139 \cdot \text{kPa}$	$P_{max} = 185 \cdot \text{kip}$	$P_{max} = 825 \cdot \text{kN}$	$Check_1 = 1$
Strength I factored bearing capacity		**Factored Sliding Resistance**		**Ultimate Bearing OK (1) or not OK (0)?**
$q_R = 3.3 \cdot \text{ksf}$	$q_R = 158.462 \cdot \text{kPa}$	$P_{fres} = 148 \cdot \text{kip}$	$P_{fres} = 660 \cdot \text{kN}$	$Check_2 = 1$

CORESTONE ENGINEERING, INC.
1345 Capital Blvd, Suite B, Reno, NV 89502

Date:	6/27/2019	
Revision No:	2019 March - 1	
Developed by:	JWP/PV	
Project Name:	**San Bernardino Class 1 Bike Trail Project**	
Calculated by:	PV	
Project Number:	**5012-02-1**	B-02 Data
Checked by:	pv	
Design Case:	**Pathway Bridge - Abutments (B=5 ft)**	

SETTLEMENT USING AASHTO-MODIFIED "HOUGH METHOD"

Only cells with blue background and blue text should be modified

AASHTO, 2007, LRFD Design Manual 4th Edition p 10-55 Same for AASHTO 2017

Hough, 1959, Compressibility as the Basis for Soil Bearing Value, Journal of the Soil Mechanics and Foundations Division, ASCE SM4, August 1959

Foundation Load	4700	kN	1056.6	kips		
Foundation Depth	0	m	0.0	ft		
Foundation Width	1.524	**m**	5	ft	**11740.1 psf**	
Foundation Length	5.4864	**m**	18	ft	For 1 inch settlement (Service Value)	
Depth of Influence (3B)	4.572	m	15.0	ft	check	0.98 inch
Depth to Water Table	1.829	m				

Depth m	Unit Weight kN/m^3	Total Stress kPa	Eff Stress kPa	Inc Stress kPa	Hough C'	Settlement m	depth ft
0.000	20.0	0	0	NA	NA	NA	0
1.829	20.0	37	37	301.1	150	0.01177	6
2.591	20.0	52	44	245.8	200	0.00311	8.5
3.353	20.0	67	52	205.0	75	0.00704	11
4.115	20.0	82	60	174.0	150	0.00301	13.5
4.877	20.0	98	68	149.7	175	0.00000	16
5.639	20.0	113	75	130.3	100	0.00000	18.5
6.401	20.0	128	83	114.5	75	0.00000	21
7.163	20.0	143	91	101.5	175	0.00000	23.5
7.925	20.0	158	99	90.7	175	0.00000	26
8.687	20.0	174	106	81.5	175	0.00000	28.5
9.449	20.0	189	114	73.7	175	0.00000	31
10.211	20.0	204	122	66.9	175	0.00000	33.5
10.973	20.0	219	130	61.1	175	0.00000	36
11.735	20.0	235	138	56.0	175	0.00000	38.5
12.497	20.0	250	145	51.5	175	0.00000	41
13.259	20.0	265	153	47.6	175	0.00000	43.5
14.021	20.0	280	161	44.1	175	0.00000	46
14.783	20.0	296	169	40.9	175	0.00000	48.5
15.545	20.0	311	176	38.1	175	0.00000	51
16.307	20.0	326	184	35.6	200	0.00000	53.5
17.069	20.0	341	192	33.3	200	0.00000	56
17.831	20.0	357	200	31.3	200	0.00000	58.5
18.593	20.0	372	207	29.4	200	0.00000	61
19.355	20.0	387	215	27.7	200	0.00000	63.5
20.117	20.0	402	223	26.1	200	0.00000	66
20.879	20.0	418	231	24.7	200	0.00000	68.5
21.641	20.0	433	238	23.3	200	0.00000	71
22.403	20.0	448	246	22.1	200	0.00000	73.5
23.165	20.0	463	254	21.0	200	0.00000	76
23.927	20.0	479	262	20.0	200	0.00000	78.5
24.689	20.0	494	270	19.0	200	0.00000	81
25.451	20.0	509	277	18.1	200	0.00000	83.5

0.00 Df, m

1.5 B, m

5.5 L, m

562.1 q, kN/m^2

0.0249 m
25 mm
0.98 inches

CORESTONE ENGINEERING, INC.
1345 Capital Blvd, Suite B, Reno, NV 89502

	Date:	6/27/2019
	Revision No:	2019 March - 1
	Developed by:	JWP/PV
Project Name: San Bernardino Class 1 Bike Trail Project	**Calculated by:**	PV
Project Number: 5012-02-1 B-02 Data	**Checked by:**	pv
Design Case: Pathway Bridge - Abutments (B=10 ft)		

SETTLEMENT USING AASHTO-MODIFIED "HOUGH METHOD"

Only cells with blue background and blue text should be modified

AASHTO, 2007, LRFD Design Manual 4th Edition p 10-55 Same for AASHTO 2017

Hough, 1959, Compressibility as the Basis for Soil Bearing Value, Journal of the Soil Mechanics and Foundations Division, ASCE SM4, August 1959

Foundation Load	3300	kN	741.9	kips	
Foundation Depth	0	m	0.0	ft	
Foundation Width	3.048	**m**	10	ft	**4121.5 psf**
Foundation Length	5.4864	**m**	18	ft	For 1 inch settlement (Service Value)
Depth of Influence (3B)	9.144	m	30.0	ft	check 0.99 inch
Depth to Water Table	1.829	m			

Depth m	Unit Weight kN/m^3	Total Stress kPa	Eff Stress kPa	Inc Stress kPa	Hough C'	Settlement m	depth ft
0.000	20.0	0	0	NA	NA	NA	0
1.829	20.0	37	37	130.1	150	0.00803	6
2.591	20.0	52	44	112.0	200	0.00209	8.5
3.353	20.0	67	52	97.5	75	0.00465	11
4.115	20.0	82	60	85.7	150	0.00196	13.5
4.877	20.0	98	68	75.9	175	0.00142	16
5.639	20.0	113	75	67.7	100	0.00212	18.5
6.401	20.0	128	83	60.8	75	0.00242	21
7.163	20.0	143	91	54.9	175	0.00089	23.5
7.925	20.0	158	99	49.8	175	0.00077	26
8.687	20.0	174	106	45.4	175	0.00067	28.5
9.449	20.0	189	114	41.6	175	0.00000	31
10.211	20.0	204	122	38.2	175	0.00000	33.5
10.973	20.0	219	130	35.2	175	0.00000	36
11.735	20.0	235	138	32.6	175	0.00000	38.5
12.497	20.0	250	145	30.2	175	0.00000	41
13.259	20.0	265	153	28.1	175	0.00000	43.5
14.021	20.0	280	161	26.3	175	0.00000	46
14.783	20.0	296	169	24.5	175	0.00000	48.5
15.545	20.0	311	176	23.0	175	0.00000	51
16.307	20.0	326	184	21.6	200	0.00000	53.5
17.069	20.0	341	192	20.3	200	0.00000	56
17.831	20.0	357	200	19.2	200	0.00000	58.5
18.593	20.0	372	207	18.1	200	0.00000	61
19.355	20.0	387	215	17.1	200	0.00000	63.5
20.117	20.0	402	223	16.2	200	0.00000	66
20.879	20.0	418	231	15.4	200	0.00000	68.5
21.641	20.0	433	238	14.6	200	0.00000	71
22.403	20.0	448	246	13.9	200	0.00000	73.5
23.165	20.0	463	254	13.2	200	0.00000	76
23.927	20.0	479	262	12.6	200	0.00000	78.5
24.689	20.0	494	270	12.0	200	0.00000	81
25.451	20.0	509	277	11.5	200	0.00000	83.5

0.00 Df, m

3.0 B, m

5.5 L, m

197.3 q, kN/m^2

0.0250 m
25 mm
0.99 inches

Project Name:	San Bernardino Class 1 Bike Trail Project	Calc By:	PV
Project No.:	5012-02-1	Check By:	PV
Design Case:	Pathway Bridge Abutments B = 5 ft	Date:	6/26/2019

AASHTO (2017) Table 10.6.3.1.2c-1 RC_{BC} Values For Footing on Slope

C' = 0 Phi = 36 deg

B/H	β		
	10	20	30
0.1	0.800	0.380	0.170
0.2	0.780	0.370	0.160
0.4	0.720	0.360	0.170
0.6	0.660	0.340	0.170
1	0.700	0.450	0.320
1.5	0.740	0.560	0.470
3	0.770	0.580	0.620

Input per the range to determine interpolated values

20 -30	10-20	Range
β	β	
20	10	input
0.380	0.800	
0.370	0.780	
0.360	0.720	
0.340	0.660	
0.450	0.700	
0.560	0.740	
0.580	0.770	

B/H 0.8

RC_{BC} = 0.404

Input bracket values based on above calcs for linear interpolation

B/H	RC_{BC}
0.6	0.340
1	0.450

RC_{BC} = 0.404 (Only for interpolation)

Project Name:	San Bernardino Class 1 Bike Trail Project	Calc By:	PV
Project No.:	5012-02-1	Check By:	PV
Design Case:	Pathway Bridge Abutments B = 10 ft	Date:	6/26/2019

AASHTO (2017) Table 10.6.3.1.2c-1 RC_{BC} Values For Footing on Slope

C' = 0 Phi = 36 deg

Input per the range to determine interpolated values

B/H	β		
	10	20	30
0.1	0.800	0.380	0.170
0.2	0.780	0.370	0.160
0.4	0.720	0.360	0.170
0.6	0.660	0.340	0.170
1	0.700	0.450	0.320
1.5	0.740	0.560	0.470
3	0.770	0.580	0.620

20 -30	10-20	Range
β	β	
20	10	input
0.380	0.800	
0.370	0.780	
0.360	0.720	
0.340	0.660	
0.450	0.700	
0.560	0.740	
0.580	0.770	

B/H 1.7

RC_{BC} = 0.562

Input bracket values based on above calcs for linear interpolation

B/H	RC_{BC}
1.5	0.560
3	0.580

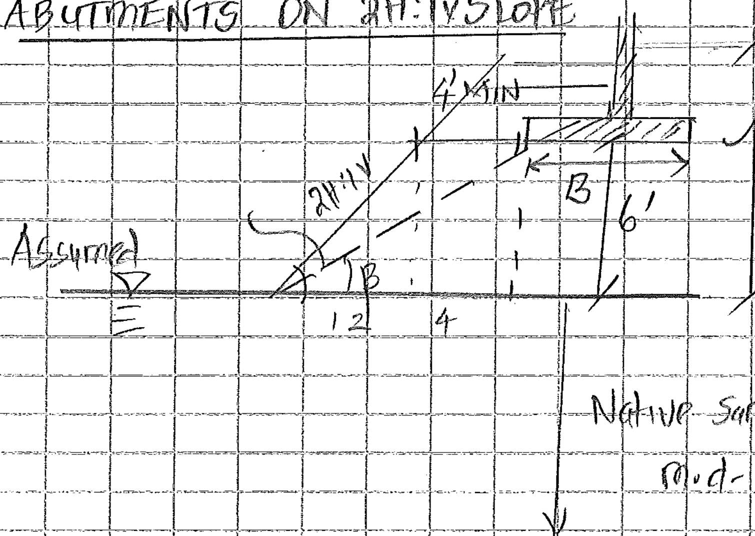
RC_{BC} = 0.562 (Only for interpolation)

CORESTONE ENGINEERING, INC
GEOTECHNICAL SERVICES & CONSTRUCTION MATERIALS TESTING
1345 CAPITAL BLVD, SUITE B, RENO, NV 89502; PH: 775-636-5916

DATE: 6/27/18 CALCULATED BY: PV CHECKED BY: V
PROJECT: SAN BERNARDINO CLASS I BIKE TRAIL PROJECT NO.: 5012-D2-1
SUBJECT: BRIDGE FOUNDATIONS

AASHTO LRFD (2017)

ABUTMENTS ON 2H:1V SLOPE



Embankment Fill
$\phi = 36^\circ$ $\gamma = 125$ pcf

10' Assumed

$\tan\beta = \frac{6}{16}$

$\beta = 20.56^\circ$

For settlement calcs, use the most conservative
USE DATA FROM B-D2 (East Side of UPPER TRUCKEE RIVER

FOR PIER FOUNDATIONS $N = 7 - 35$ (VARIES)
CALC AVERAGE = $N_{160} = 13 - 48$

$(N_{1,60}) = 35$ ~ From AASHTO (2017)
Through 2B Depth $\phi_{avg} = 36^\circ$ – use 36°
(see separate calcs)

Abutments $B = 6' \rightarrow R_{CB} = 0.34$ for no slope
$N_{\gamma\,slope} = 37.752 \times 0.404 = 15.25$
$B = 10' \rightarrow R_{CB} = 0.562 \times 37.752$
$\cong 21.22$

Granular Soils Friction Angle - AASHTO (2017) Table 10.4.6.2.4.1

$N1_{60}$	ϕ_f (deg)	
	low	high
4	27	32
5	27.5	32.5
6	28	33
7	28.5	33.5
8	29	34
9	29.5	34.5
10	30	35
11	30.25	35.25
12	30.5	35.5
13	30.75	35.75
14	31	36
15	31.25	36.25
16	31.5	36.5
17	31.75	36.75
18	32	37
19	32.25	37.25
20	32.5	37.5
21	32.75	37.75
22	33	38
23	33.25	38.25
24	33.5	38.5
25	33.75	38.75
26	34	39
27	34.25	39.25
28	34.5	39.5
29	34.75	39.75
30	35	40
31	35.15	40.15
32	35.3	40.3
33	35.45	40.45
34	35.6	40.6
35	35.75	40.75
36	35.9	40.9
37	36.05	41.05
38	36.2	41.2
39	36.35	41.35
40	36.5	41.5
41	36.65	41.65
42	36.8	41.8
43	36.95	41.95
44	37.1	42.1
45	37.25	42.25
46	37.4	42.4
47	37.55	42.55
48	37.7	42.7
49	37.85	42.85
50	38	43

Values from AASHTO Table

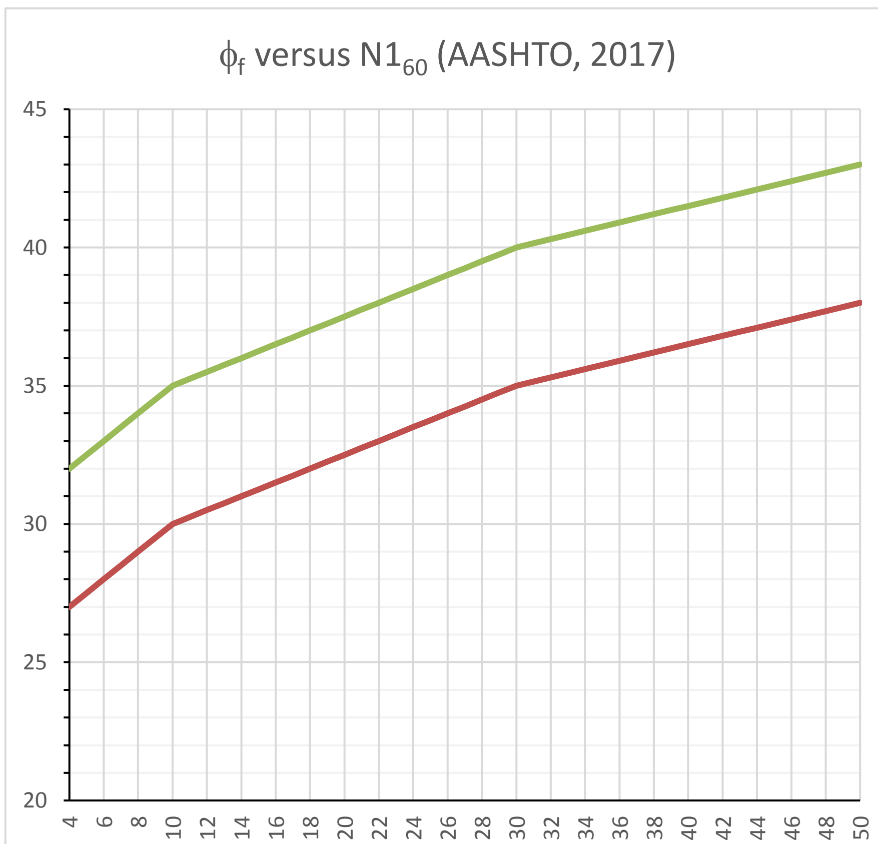
Interpolated values

Input

$N1_{60}$ = 35

Output

ϕ_f (deg)		
35.75	**40.75**	**38.25**
low	high	average



Project Name:	San Bernardino Class 1 Bike Trail Project	Developed By:	PV
Project No:	5012-02-1	Calculated By:	PV
Description:	Retaining Walls	Checked By:	
		Date:	6/28/2019

Inclination of active failure plane and lateral earth pressure coefficients

Reference: 1. Geotechnical Earthquake Engineering, FHWA HI-99-012, Dec 1998 (GEE)
2. Earth Retaining Structures, NHI Course No. 13236, May 1998 (ERS)

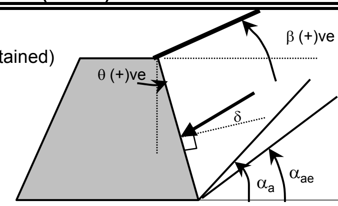
ϕ = 36 deg 0.628 rad. (Friction angle of soil retained)

β = 0 deg 0.000 rad.

δ = 12 deg 0.209 rad. (ϕ/3 ?)

θ = 0 deg 0.000 rad.

$\phi/3$ = 12.0 deg



use (-)ve values as appropriate

A = 0.42 (Design acceleration coeff.) k_h = 0.21

(Sds/2.5 - geo report) k_v = 0 (generally zero)

$\Psi = \tan^{-1}[k_h / (1-k_v)]$ = 11.86 deg 0.207 rad.

Failure Wedge (Static and Seismic)

From Mononobe-okabe theory, (GEE 9-30)

$$\alpha_{ae} = \phi - \psi + \arctan\left(\frac{\sqrt{F_1(F_1+F_2)(1+F_3F_1)} - F_1}{1+F_3(F_1+F_2)}\right)$$

α_{ae} = 43.1 deg

(Seismic Wedge)

Where,

		when $\psi = 0$ deg,
F1 = tan ($\phi-\psi-\beta$) =	0.448	0.727
F2 = cot ($\phi-\psi-\theta$) =	2.231	1.376
F3 = tan ($\delta+\theta+\psi$) =	0.442	0.213

When $\psi = 0$,

α_a = 58.6 deg

(Static ?)

Compare with Rankine active failure angle (static loading with horizontal backfill)

$\alpha_a = 45 + \phi/2$ → **α_a = 63.0 deg**

(use this for static wedge)

Results in Equivalent Fluid Presssure (pcf)		
Unit weight =		125 pcf
Case	Static	Dynamic
At-rest	52	N/A
active	**30**	**47**
passive	481	

Lateral Earth Pressure Coefficients (Static and Seismic):

Using Coulomb's Theory, (ERS 2-4)

$$K_a = \frac{\cos^2(\phi-\theta)}{\cos^2\theta\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$

K_a = 0.240

Check:

Rankine $K_a = \tan^2(45 - \phi/2)$ = 0.260

(Only for vertical walls with level backfill)

Using Mononobe-Okabe Theory, (GEE 9-13b)

$$K_{ae} = \frac{\cos^2(\phi-\psi-\theta)}{\cos\psi\cos^2\theta\cos(\delta+\theta+\psi)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\psi-\beta)}{\cos(\delta+\theta+\psi)\cos(\beta-\theta)}}\right]^2}$$

K_{ae} = 0.374

Further,

Coulomb K_p = 6.080 (use only when $\delta \le \phi/3$) Mononobe-Okabe, K_{pe} = 5.187

Rankine K_p = 3.852 (vertical wall with level backfill)

NAVFAC chart can also be used to determine K_p & K_{pe} values (more reasonable values for some cases).

Note : Use WASP to calculate K_{ae} when Mononobe-Okabe equation fails or for special cases.