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SECTION H

GEOTECHNICAL EXPLORATION REPORT

CALICO GHOST TOWN BRIDGE REPLACEMENT PROJECT

FOR

SAN BERNARDINO COUNTY REGIONAL PARK
SAN BERNARDINO, CALIFORNIA

PROJECT NO.: 1378.0160

GEOTECHNICAL EXPLORATION

PROPOSED CALICO GHOST TOWN BRIDGE REPLACEMENT PROJECT

36600 GHOST TOWN ROAD, YERMO

UNINCORPORATED SAN BERNARDINO COUNTY CALIFORNIA

PROJECT SERVICE REQUEST #SD004

Prepared For

SAN BERNARDINO COUNTY DEPARTMENT OF PUBLIC WORKS - SPECIAL DISTRICTS
222 WEST HOSPITALITY LANE, SECOND FLOOR
SAN BERNARDINO, CALIFORNIA 92415

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Project No. 038.0000020706

January 19, 2024

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San Bernardino County
Department of Public Works – Special Districts
222 West Hospitality Lane, Second Floor
San Bernardino, California 92415-0450

Attention: Mr. Russel Viloria
Project Manager

Subject: Geotechnical Exploration
Proposed Calico Ghost Town Bridge Replacement Project
36600 Ghost Town Road, Yermo
Unincorporated San Bernardino County, California
Project Service Request #SD004

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has performed this geotechnical exploration in support of the proposed Calico Ghost Town Bridge Replacement Project (Project Service Request #SD004). The purpose of our study was to evaluate the subsurface geotechnical conditions with respect to the proposed improvements and to provide geotechnical recommendations for design and construction of the proposed prefabricated steel pedestrian bridge replacement project for the San Bernardino County Regional Parks Department.

Based on this study, construction of the proposed prefabricated steel pedestrian bridge replacement is considered feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during design and construction. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, corrosive soils, and difficulty of excavation due to the underlying very dense bedrock conditions.

We appreciate the opportunity to be of service for this project. If you have any questions or concerns, please contact us at your convenience. The undersigned can be reached at **(866) *LEIGHTON***.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Description

The project site is located within the Calico Ghost Town Regional Park at 36600 Ghost Town Road, north of the Yermo area in unincorporated San Bernardino County, California. The site location (34.9508°N Latitude, -116.8649°W Longitude) and immediate vicinity are shown on Figure 1, *Site Location Map*.

The site is within the Calico Mountains near the northern end of Calico Ghost Town Regional Park. The park is located approximately 2.5 miles north of Interstate Highway 15 (I-15). Based on review of aerial imagery, the site is currently developed and contains several historic structures dating back to the early 1880s. We understand the existing 65-foot-long wooden pedestrian bridge will be replaced with a new prefabricated steel pedestrian bridge and associated improvements.

Based on review of available topographic maps, site elevations (El.) range from approximately El. 2,260 to El. 2,350 feet above mean sea level (msl). The ephemeral stream that the existing bridge spans drains gently towards the southwest.

1.2 Proposed Development

Our understanding of this project is based on the provided *Project Service Request #SD004 for the Calico ghost Town – Bridge Replacement Project*, dated November 21, 2023. We understand that the San Bernardino County Department of Public Works, Special District Department plans to replace the existing wooden pedestrian bridge connecting the main park to the existing Calico Ghost Town Fan Club structure with a new prefabricated steel pedestrian bridge. The newly prefabricated steel bridge will also include associated improvements such as bridge abutments, guardrails, bollards, and an ADA compliant concrete ramp.

Structural loading and preliminary structural plans of the proposed prefabricated bridge have not been provided to us at the time of this report. We assume that the prefabricated steel bridge will be relatively lightly loaded.

1.3 Purpose and Scope

The purpose of our work was to evaluate the subsurface conditions at the site relative to the proposed development and provide preliminary geotechnical recommendations to aid in project planning. The scope of this evaluation included the following tasks:

- *Background Review* – We reviewed readily available reports, literature, aerial photographs, and maps relevant to the site available from our in-house library or in the public domain. We evaluated geological hazards and potential geotechnical issues that may significantly affect the site. The documents reviewed are listed in Appendix A, *References*.

- *Site Reconnaissance* – We performed a visual site reconnaissance to mark the locations proposed for hollow-stem auger test borings and to assess access throughout the site. Once the locations were marked, DigAlert (811) was notified for utility clearance. The services of a private utility locator were also retained in an effort to identify any private utility lines that were not marked by DigAlert and possibly in conflict with our proposed boring locations.

- *Field Exploration* – Field exploration was performed on December 27, 2023 and consisted of two (2) hollow-stem auger borings for geotechnical logging and sampling (designated as LB-1 and LB-2). Geotechnical borings were drilled to depths of 30.5 feet and 20 feet below ground surface (bgs), respectively. The approximate locations of the borings are shown on Figure 2, *Exploration Location Map.* Logs of the exploration are included in Appendix B, *Exploration Logs.*

  During advancement of the hollow-stem auger borings, bulk samples and drive samples were obtained for geotechnical laboratory testing. Drive samples were collected using a Modified California ring-lined sampler with sampling conducted in accordance with ASTM Test Method D 3550 and by the Standard Penetration Test (SPT) method in accordance with ASTM Test Method D 1586 within the hollow-stem auger borings. The ring and SPT samplers were driven for a total penetration of 18 inches using a 140-pound automatic hammer falling 30 inches. The number of blows per 6 inches of penetration was recorded on boring logs, see Appendix B, *Exploration Logs.* Bulk samples were collected from the upper 5 feet.

The borings were logged in the field by a member of our technical staff under the supervision of a State of California licensed Certified Engineering Geologist. Each soil sample collected was reviewed and described in general accordance with the Unified Soil Classification System. The samples were sealed and packaged for transportation to our in-house laboratory.

- *Laboratory Testing* – Geotechnical laboratory testing was performed on selected soil samples collected during our field exploration to determine engineering properties of encountered subsurface soils. The results of laboratory testing are presented in Appendix C, *Laboratory Test Data.*

- *Engineering Analysis* – Geotechnical analysis was performed on the collected and available data to develop conclusions and preliminary recommendations for design and construction of the improvements as currently planned.

- *Report Preparation* – Results of our geotechnical study have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed prefabricated steel pedestrian bridge replacement as currently planned.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geology

The site is located in the Mojave Desert Geomorphic Province of southern California. This geomorphic province is bounded to the north and northwest by the Garlock fault and to the southwest by the San Andreas Fault. The central Mojave Desert, where this project is located, has a history of crustal deformation that includes extension, contraction, and lateral faulting.

This project is located within the Calico Mountains, which are mostly composed of Miocene sedimentary and volcanic rocks. The Calico section of the Calico-Hidalgo fault zone has been mapped to trace approximately 0.2 mile south of the site. Lacustrine deposits north of the Calico section have been folded, indicating that the Calico Mountains have experienced compressional deformation during the period between approximately 23.0 to 2.6 million years ago.

The site and surrounding vicinity are underlain by Tertiary volcanoclastic rocks. Figure 3, *Regional Geology Map,* presents the site location in relation to the predominate geologic materials (volcanic and sedimentary rocks) of the area. Figure 4, *Regional Fault and Historical Seismicity Map,* presents the site location in relation to active faults and epicenters of relatively large (> M_w 4.0) historical earthquakes.

2.2 Subsurface Conditions

Based upon our review of geologic maps and our subsurface exploration, the site is underlain by volcanoclastic rock (see Figure 3, *Regional Geology Map*). Within our borings, a thin mantle of topsoil was encountered consisting of gravel underlain by silty sands and clayey sands with varying amounts of gravel approximately 1 to 2 feet thick.

Volcanoclastic rock encountered below the topsoil within our borings was disturbed during drilling, but appeared to contain silt to sand sized volcanic fragments. Rock encountered in our borings was considered moderately soft where drilling and sampling was achieved without much difficulty, moderately hard where drilling was more difficult and only partial penetration of samplers with 50 blows with the drill rig's auto hammer was possible, and hard where drilling refusal was met. We encountered drilling refusal at depths ranging from 20 to 30½ feet bgs in our two borings.

In situ moisture contents of the recovered bedrock samples within the upper 10 feet ranged from 2 to 5 percent. More detailed descriptions of the subsurface conditions encountered are presented on the boring logs (Appendix A).

2.3 Groundwater Conditions

Groundwater was not encountered in any of our borings, which extended to a maximum depth of approximately 30.5 feet bgs. The volcanoclastic rock onsite is not generally considered water bearing. The historically highest groundwater in the vicinity has been measured from nearby wells located within alluvial deposits (State Well Nos. 10N01E22C001S and 10N01E27C001S) to have been at elevations that correlate to levels roughly 300 to 400 feet below the existing ground surface at the project site. Due to the presence of shallow rock at the site, rainfall runoff may collect in the drainage under the proposed bridge and seasonal groundwater perched on the relatively impermeable rock may be encountered during construction.

2.4 Engineering Properties

Geotechnical engineering properties determined to be relevant for the proposed development were evaluated on the basis of field observations, laboratory testing and review of the interpreted subsurface profile and engineering correlations. The following summarizes the relevant properties evaluated for this project.

2.4.1 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of foundations could result.

Expansion Index (EI) testing was performed on a representative sample obtained from near-surface soils within LB-2. Results of Expansion Index testing indicate near surface soil will exhibit a very low expansion potential.

2.4.2 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) provisions, adopted by the 2022 CBC (CBC, 2022 and ACI, 2014).

A sulfate test performed during this study of a near-surface soil sample onsite resulted in a sulfate content of 300 ppm (less than 0.1 percent by weight), indicating negligible sulfate exposure (Exposure Class S0).

2.4.3 Resistivity, Chloride, and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content, and pH level. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive, while soil having a minimum resistivity of 1,000 to 2,000 is considered corrosive. Soil with a chloride content of 500 ppm or greater is considered corrosive to ferrous metals.

For screening purposes, a bulk sample of representative near-surface soils from boring LB-1 was tested to determine minimum resistivity and chloride content. The test results indicated a minimum resistivity of 1,950 ohm-cm, a chloride content of 300 ppm, and a pH value of 9.71. Based on these results, onsite soils are expected to be corrosive to ferrous metals per ASTM STP 1013.

2.4.4 Soil Compressibility and Collapse

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on our investigation, near-surface native soil encountered is generally considered slightly to moderately compressible. Partial removal and recompaction of this material under shallow foundations will help reduce the potential for adverse total and differential settlement of the proposed improvements. The onsite bedrock is not considered compressible.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Based on the relatively shallow bedrock underlying the site, and after remedial earthwork is completed, the onsite soils are anticipated to have negligible collapse potential.

2.5 Surface Fault Rupture

The state of California passed the Alquist-Priolo Earthquake Fault Zoning (AP) Act into law following the February 9, 1971, M_W=6.6 San Fernando earthquake. The AP Act provides a mechanism for reducing potential losses from surface fault rupture on a statewide basis. The intent of the AP Act is to enhance public safety by prohibiting the siting of most structures for human occupancy across known traces of active faults that constitute a potential hazard to structures from active surface faulting. The fault classification criteria adopted by the California Geological Survey (CGS) defines Earthquake Fault Zones along active faults. An active fault is defined as one that has ruptured during Holocene time (the last 11,700 years).

No State of California or County of San Bernardino Earthquake Fault Zones have been mapped within or projecting towards the site.

The closest mapped potentially active fault to the site is the Calico section of the Calico-Hidalgo fault zone, located approximately 0.2 mile southwest of the subject site. Based on the absence of faults known or mapped across the project, the lack of tonal lineaments or other geomorphic indicator of fault activity, the potential for fault ground rupture at the site is considered low. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historical Seismicity Map*.

2.6 Seismicity and Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California (see Figure 4, *Regional Fault and Historical Seismicity Map*). The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics.

The site will experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2022 CBC:

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude (degrees):	34.9508, -117.8649
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	B
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), $\boldsymbol{S_S}$	1.769 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), $\boldsymbol{S_1}$	0.612 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), $\boldsymbol{F_a}$	0.9
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), $\boldsymbol{F_v}$	0.8
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), $\boldsymbol{S_{MS}}$	1.592 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), $\boldsymbol{S_{M1}}$	0.49 g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), $\boldsymbol{S_{DS}}$	1.061 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), $\boldsymbol{S_{D1}}$	0.326 g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.794 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), $\boldsymbol{F_{PGA}}$	0.9
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), $\boldsymbol{PGA_M}$	0.715 g

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 6.28 (M_W) at a distance on the order of 1.56 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), and corresponding peak ground acceleration of 0.66 g.

2.7 Liquefaction Potential

Liquefaction is the loss of soil strength and stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged, and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The State of California has not evaluated liquefaction hazards for the quadrangle the site. The San Bernardino County Geologic Hazard Overlay Map EI02 Yermo indicates that the site is outside any zone of liquefaction susceptibility (San Bernardino County, 2007).

Based on the shallow, very dense, relatively impermeable bedrock encountered at the site, and lack of shallow groundwater, the subsurface soils and bedrock are not considered susceptible to liquefaction.

2.8 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed (1987), and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Design/historic high groundwater levels of 300 feet below ground surface were used in the analysis. Based on our analysis, and shallow bedrock materials encountered during our exploration, the potential seismic induced settlement is nil. Results of our seismic settlement analysis are presented in Appendix D.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed bridge is considered feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during design and construction. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, corrosive soils, and difficulty of excavation due to the underlying very dense bedrock conditions.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to evaluate the effect upon the recommendations presented herein. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the County of San Bernardino, and other governing agencies.

3.1 General Earthwork and Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. Overexcavation and recompaction recommendations are presented in the following paragraphs. The General Earthwork and Grading Recommendations are included in Appendix E. In case of conflict, the following recommendations shall supersede those provided in Appendix E.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. Any underground obstructions onsite interfering with the proposed construction should be removed or rerouted to preserve their function. Resulting cavities should be properly backfilled and compacted. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable dry fill deposits by a representative of the geotechnical engineer.

3.1.2 Overexcavation and Recompaction

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade should be prepared in such a manner that a uniform response to the applied loads is achieved.

All compressible topsoil should be removed during foundation grading should be removed to a minimum depth of 3 feet below current grades or 2 feet below bottom of footings, whichever is greater. Deeper overexcavation may be recommended, depending on exposed conditions during grading. Removal bottoms should extend horizontally a minimum of 3 feet beyond the outside edges of footings, or a distance equal to the depth of overexcavation below the footings, whichever is greater. Suitability of all removal bottoms should be reviewed and evaluated by a representative of Leighton.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and site walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be evaluated for suitability. Once determined geotechnically acceptable, the subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

All fills should be observed and tested in the field by a Leighton representative prior to fill placement or foundation or pavement construction to ensure adequate moisture conditioning and compactive requirements are met. Fill soil should be compacted to a minimum of 90 percent relative compaction per ASTM D1557. The upper 6 inches of subgrade soil in pavement areas and aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.3 Fill Placement and Compaction

Onsite soil may be used for compacted structural fill provided it is free of debris, organic material and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and tested by Leighton as needed or required.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 Import Fill Soil

The geotechnical parameters of any import soil should be evaluated and accepted by Leighton prior to use as fill on the site. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples for geotechnical and analytical testing for potential chemicals of concern. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.5 Shrinking and Bulking

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. This value does not factor in removal of debris or other materials. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as what occurs during processing an overexcavation (subgrade) bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 5% +/- 3%
Subsidence (overexcavation bottom processing)	Approximately 0 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.6 Rippability and Oversized Material

Any oversized material (rock or rock fragments greater than 8 inches in dimension) should be placed outside the limits of structural fills (i.e. not in foundation areas). Resistant rock underlies the site and excavations into this unit will be made with difficulty. Heavy ripping should be expected where hard bedrock was encountered at depths as shallow as 2 feet bgs.

Based on the conditions observed during drilling and at the surface, we anticipate that considerable quantities of rock fragments and oversized rock will be generated during excavation. Therefore, rock disposal during excavation will be needed. We recommend that, where possible, large rocks be incorporated into nonstructural fills. If sufficient nonstructural fill areas are not available, size reduction processing or off-site disposal may be required.

The amount of excavation into resistant bedrock can be minimized with a design that minimizes deep cuts into bedrock.

3.2 Shallow Foundation Recommendations

The proposed prefabricated building can be supported on conventional spread or strip footing shallow foundation systems. Maximum column loading and wall loading is not available at the time of this report. We have anticipated the prefabricated structure will be lightly loaded and consist of steel components. Structural loading information should be provided to us when available for review.

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a “very low” expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment of 24 inches, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 3,000 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 4,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.4. The passive resistance may be computed using an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction — Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.2.5 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.3 Exterior Concrete Flat Work

Exterior concrete slabs-on-grade should have a minimum thickness of 4 inches. Common Type II cement should be adequate for concrete flatwork not exposed to recycled water. Concrete flatwork should be placed on previously compacted fill. If fill material has been disturbed or becomes dry, the subgrade soil should be scarified to a minimum depth of 18 inches, moisture conditioned to 2 percentage points above the optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

Exterior concrete ramps, curbs, gutters, and sidewalks often crack. Inclusion of joints at frequent intervals and reinforcement will help control the locations of the cracks, and thus reduce the unsightly appearance. Appropriate joints or saw cuts should be constructed in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. If cracking occurs, repairs may be needed to mitigate the trip hazard and/or improve the appearance.

3.4 Retaining Wall Recommendations

Areas planned for retaining walls should be over-excavated in accordance with the recommendations provided in Section 3.1. Retaining walls should be backfilled with *very low* expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 5, *Retaining Wall Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls up to 6 feet tall; taller walls should be checked on a case-by-case basis:

Static Equivalent Fluid Weight (pcf)	
Condition	**Level Backfill**
Active	40
At-Rest	60
Passive (allowable)	250 (Maximum of 3,000 psf)

The above values do not contain an appreciable factor of safety, except the allowable passive contains a factor of safety of 2, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.55 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

We recommend that the wall designs for walls 6 feet tall or taller be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 22 pcf of level backfill, which is added to the equivalent fluid pressure.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

Retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 12 inches. An allowable bearing capacity of 2,500 pcf may be used for retaining wall footing design, based on the minimum footing width and depth.

3.5 Geochemical and Resistivity Characteristics

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2014). ACI provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm).

Laboratory test results indicated that onsite soils at shallow depth have “negligible” soluble sulfate content (per Section 4.3 of ACI 318). Concrete structures in contact with the on-site soils may be designed for **negligible** sulfate exposure in accordance with ACI 318 (ACI, 2014). If the concrete is expected to be in contact with reclaimed water, Type V cement and a water/cement ratio of 0.45 should be used.

Resistivity: The results of the resistivity test indicated that the underlying soil is **corrosive** to buried ferrous metals per ASTM STP 1013.

As a general mitigation measure, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous pipe when possible. Ferrous pipe can also be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate the pipe from on-site soils. Once plans are developed and the type of pipe is known, i.e. steel piping, iron piping, copper tubing etc. additional recommendations can be provided as necessary from a corrosion engineer. Plastic and vitrified clay piping do not warrant any special precautions. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with site soils possessing negligible sulfate reactions.

3.6 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 40 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the “competent person” required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the project Geotechnical Engineer or Certified Engineering Geologist should be maintained to facilitate construction while providing safe excavations.

3.7 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction,* (SSPWC, “Greenbook”), 2018 Edition. Utility trenches may be backfilled with onsite material, provided it is free of rubble, debris, organic and oversized material (greater than 3 inches for trench backfill within 3 feet of a pipe, and 6 inches for trench backfill above).

Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater and allows water to freely permeate. We recommend that open-graded crushed rock or similar material not be used as bedding material, unless special provisions are implemented to limit the migration of surrounding soil into the open-graded material, including surrounding the open-graded material with filter fabric (Mirafi 140N or equivalent), or mixing sand with the open-graded material. The bedding/shading material should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or in areas where the trench walls and bottom soil have a minimum sand equivalent of 15, the bedding sand may be jetted. Bedding sand should be placed in accordance with the Standard Specifications for Public Works Construction – Greenbook (Public Works Standard, Inc.), current edition.

The native soil fill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction based on ASTM D1557. The thickness of layers should be based on the compaction equipment used in accordance with the current Greenbook.

3.8 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can lead to settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved surfaces.

3.9 Pavement Design Parameters

Flexible Pavements: Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed R-value of 50, flexible pavement sections may consist of the following for the Traffic Index indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided upon completion of grading.

Asphalt Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Base Course (inches) CAB
5.0	3.0	4.0
6.0	3.5	4.0
7.0	4.0	4.5

Notes: CAB – Crushed Aggregate Base Course; Caltrans Class 2, Section 26 or SSPWC Section 200-2.2

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

Rigid Pavements: For onsite Portland Cement Concrete (PCC) pavement in truck drive aisles and truck parking areas, we recommend a minimum of 6-inch-thick concrete, placed on compacted fill subgrade, with the upper 8 inches compacted to a minimum of 95 percent relative compaction. In areas with car traffic only, we recommend a minimum of 5-inch-thick concrete, placed on compacted fill subgrade with the upper 8 inches compacted to a minimum of 95 percent relative compaction.

The PCC pavement sections should be provided with crack-control joints spaced no more than 15 feet or 10 feet on center each way for 7-inch-thick and 5-inch-thick PCC, respectively. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement.

Other Pavement Recommendations: If pavement areas are adjacent to heavily watered landscape areas, some deterioration of the subgrade load bearing capacity may result. Moisture control measures such as deepened curbs or other moisture barrier materials may be used to prevent the subgrade soils from becoming saturated. The use of concrete cutoff or edge barriers should be considered when pavement is planned adjacent to either open (unfinished) or irrigated landscaped areas. All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.10 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from this limited subsurface exploration and limited laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Leighton Consulting, Inc. should review the site foundation, grading, retaining wall and landscape plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of lot reconditioning operations and asphalt and base placement up to final asphalt capping. Our conclusions and recommendations should be reviewed and verified by Leighton Consulting Inc. during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations.

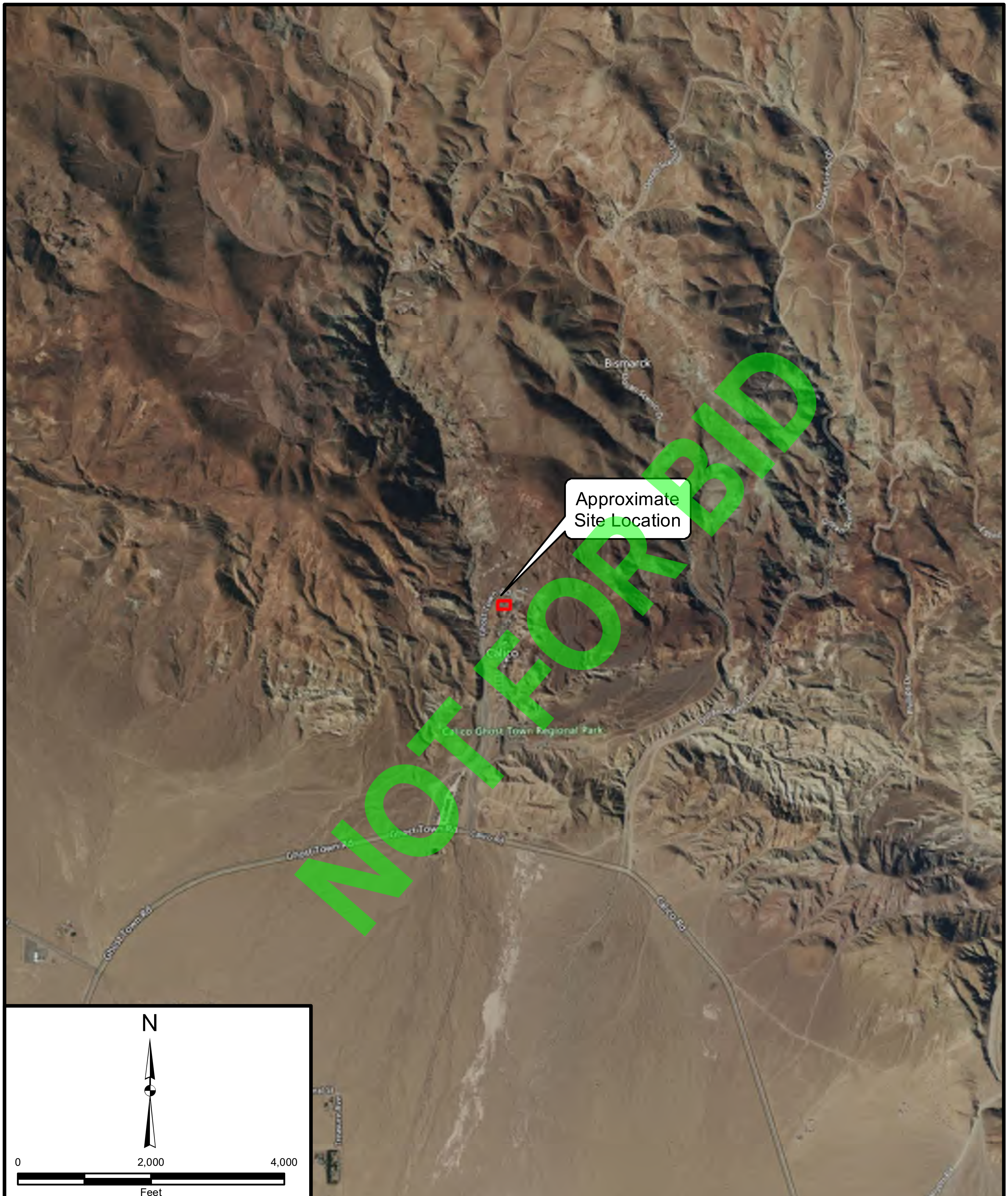
Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During over excavation of site soils
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This investigation was performed with the understanding that the subject site is proposed as and is currently a regional park development. The client is referred to Appendix F regarding important information provided by the Geo-Professional Business Association (GBA) on geotechnical engineering studies and reports and their applicability.

This report was prepared for San Bernardino County Department of Public Works based on their needs, directions, and requirements at the time of our investigation. This report is not authorized for use by and is not to be relied upon by any party except San Bernardino County Department of Public Works and its successors and assignees as owner of the property, with whom Leighton Consulting, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton Consulting, Inc.



Project: 20706	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: January 2024

Reference: © 2023 Microsoft Corporation © 2023 Maxar ©CNES (2023) Distribution Airbus DS ©

SITE LOCATION MAP

Proposed Calico Ghost Town Bridge Replacement
36600 Ghost Town Road
City of Yermo, San Bernardino County, California

FIGURE 1

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Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\20706 - Calico Ghost Town Bridge Replacement\GIS\Maps\20706_F01_SLM_2024-01-05.mxd on 12/7/2023 11:14:46 AM Author: KVM (btran)

Project: 20706	Eng/Geol: JDH/SGO
Scale: 1 " = 50 '	Date: January 2024

Service Layer Credits: © 2023 Microsoft Corporation © 2023 Maxar ©CNES (2023) Distribution Airbus DS © 2023 TomTom

Author: (btran)

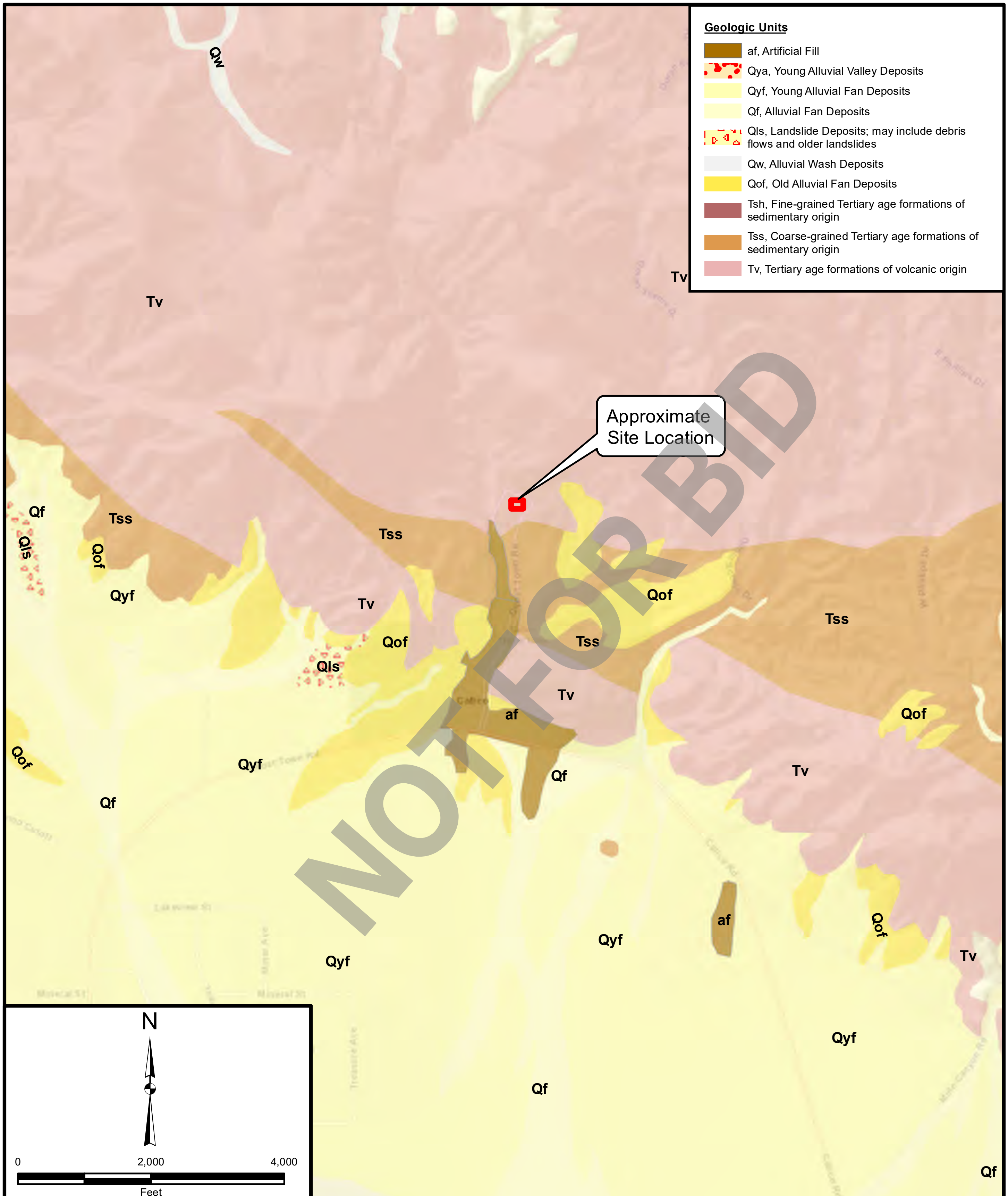
BORING LOCATION MAP

Proposed Calico Ghost Town Bridge Replacement
36600 Ghost Town Road
City of Yermo, San Bernardino County, California

FIGURE 2

Leighton

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\20706 - Calico Ghost Town Bridge Replacement\GIS\Maps\20706_F02_BLM_2024-01-05.mxd on 1/5/2024 10:03:27 AM



Project: 20706	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: January 2024

Reference: Southern California USGS Geology in GIS Format served by California Geological Survey, 2018.

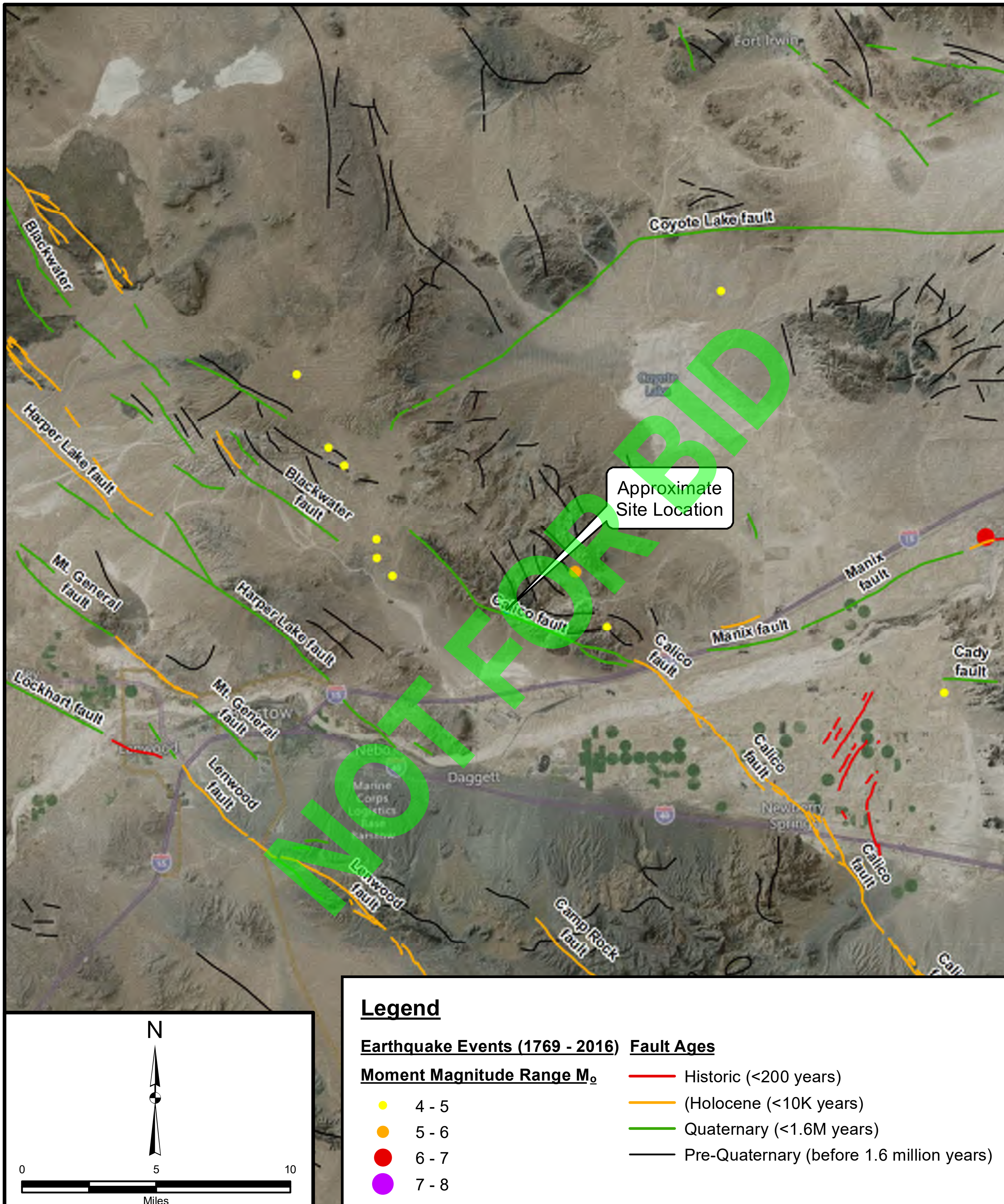
REGIONAL GEOLOGY MAP

Proposed Calico Ghost Town Bridge Replacement
36600 Ghost Town Road
City of Yermo, San Bernardino County, California

FIGURE 3

Leighton

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\20706 - Calico Ghost Town Bridge Replacement\GIS\Maps\20706_F03_RGM_2024-01-05.mxd on 1/5/2024 10:39:33 AM (btran)



Project: 20706	Eng/Geol: JDH/SGO	**REGIONAL FAULT AND HISTORICAL SEISMICITY MAP**	**FIGURE 4**
Scale: 1 " = 5 miles	Date: January 2024	Proposed Calico Ghost Town Bridge Replacement 36600 Ghost Town Road City of Yermo, San Bernardino County, California	Leighton

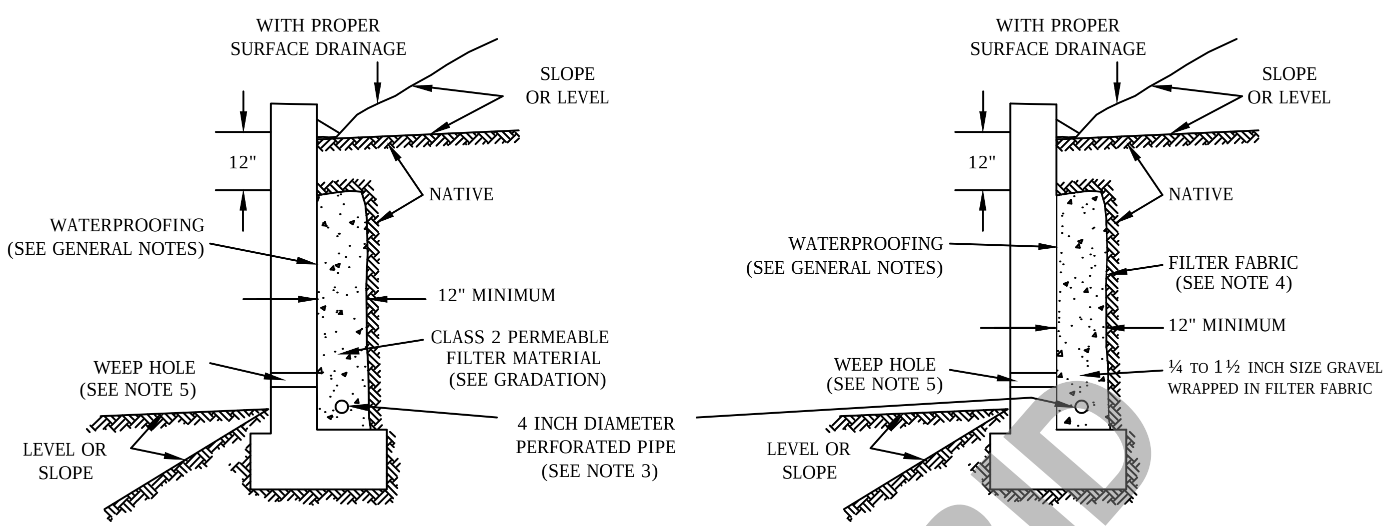
Reference: ESRI ArcGIS Online 2024
Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

Map Saved as Z:\Project Files\SA-TZ\SanBernardinoCo\20706 - Calico Ghost Town Bridge Replacement\GIS\Maps\20706_F04_RF&HSM_2024-01-05.mxd on 1/5/2024 10:35:10 AM

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 5

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000020706	**Date Drilled**	12-27-23
Project	Calico Ghot Town Bridge Replacement	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer	**Ground Elevation**	2331'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	AA

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				GP (SM)g	@Surface: Poorly Graded GRAVEL with SAND (GP), pink, dry, fine to coarse gravel, trace of cobbles (6-8 inches), 40% fine to coarse sand, 5% fines (field estimate) @0.5': SILTY SAND with GRAVEL (SM)g, fine to coarse gravel, trace cobbles, 53% sands, 23% fines (lab)	SA
2330				S-1	50/6"		2		**TERTIARY VOLCANICS (Tv)** @2.5': VOLCANOCLASTIC ROCK; pink, moderately soft, slightly moist,silt to coarse sand sized fragments -Auger grinding	
2325	5			S-2	50/6"		3		@5': VOLCANOCLASTIC ROCK; pink, moderately soft; slightly moist, silt to coarse sand fragments -Auger grinding below sample	
				S-3	50/1"				@7.5': VOLCANOCLASTIC ROCK; white/gray, moderately hard, slightly moist, very fine sand sized fragments -Auger grinding	
2320	10			S-4	50/6"		4		@10': VOLCANOCLASTIC ROCK; pink, moderately soft, slightly moist, silt to coarse sand sized fragments with trace gravel sized clasts -Auger grinding below sample	
2315	15			S-5	50/2"				@15': VOLCANOCLASTIC ROCK; pink, moderately hard, slightly moist, silt to coarse sand fragments	
2310	20			S-6	50/1"				@20': NO RECOVERY; Auger grinding; From shoe of sampler: VOLCANOCLASTIC ROCK; pink, fresh, moderately hard, slightly moist, silt to coarse sand sized fragments	
2305	25			S-7	50/1"				@25': VOLCANOCLASTIC ROCK; pink, moderately hard, slightly moist, silt to coarse sand sized fragments -Auger grinding	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-1

Project No.	038.0000020706	**Date Drilled**	12-27-23
Project	Calico Ghot Town Bridge Replacement	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer	**Ground Elevation**	2331'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	AA

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	30			S-8	50/6"				@30': VOLCANOCLASTIC ROCK; pink, moderately hard, slightly moist, silt to coarse sand sized fragments	
2300									**TOTAL DEPTH = 30.5 FEET (REFUSAL)** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED TO SURFACE WITH SOIL CUTTINGS**	
	35									
2295										
	40									
2290										
	45									
2285										
	50									
2280										
	55									
2275										
	60									

NOT FOR BID

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-2

Project No.	038.0000020706	**Date Drilled**	12-27-23
Project	Calico Ghot Town Bridge Replacement	**Logged By**	AA
Drilling Co.	2R Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer	**Ground Elevation**	2327'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	AA

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0			B-1				GP SC	@Surface: Poorly Graded GRAVEL with SAND (GP), pinkish gray, dry, fine to coarse gravel, trace of cobbles (6 inches), 20% fine to coarse sand, 5% fines (field estimate)	CR, EI, MD, RV
									@0.5': CLAYEY SAND with GRAVEL (SC), pinkish gray, fine to coarse gravel, 40% fines (field estimate)	
2325										
				R-1	50/6"		3		**TERTIARY VOLCANICS (Tv)** @2.5': VOLCANOCLASTIC ROCK; pink, moderately soft, slightly moist, silt to coarse sand sized fragments	
	5			S-1	50/6"		4		@5': VOLCANOCLASTIC ROCK; pink, moderately soft, slightly moist, silt to coarse sand sized fragments -Auger grinding below sample	
2320										
				R-2	50/1"				@7.5': VOLCANOCLASTIC ROCK; light pink, moderately hard, slightly moist, silt to coarse sand sized fragments	
	10			S-2	50/6"		5		@10': VOLCANOCLASTIC ROCK; light pinkish gray, moderately soft, slightly moist, silt to coarse sand with some gravel sized clasts	-200
2315										
	15			R-3	50/2"				@15': VOLCANOCLASTIC ROCK; white/ gray, moderately hard, slightly moist, silt to coarse sand fragments with angular fine to coarse gravel sized clasts (breccia)	
2310										
	20			S-3	50/1"				@20': VOLCANOCLASTIC ROCK; white/ gray, moderately hard, slightly moist, silt to coarse sand sized fragements with angular fine to coarse gravel sized clasts (breccia)	
2305									**TOTAL DEPTH = 20 FEET (REFUSAL)** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED TO SURFACE WITH SOIL CUTTINGS**	
	25									
2300										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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APPENDIX C

LABORATORY TEST DATA

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Calico Ghost Town Bridge Replacement	Tested By:	RMC/KJ	Date:	01/04/24
Project No.:	38.0000020706	Checked By:	A. Santos	Date:	01/09/24
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Reddish brown clayey sand with gravel (SC)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

			Scalp Fraction (%)	
Preparation Method:	X	Moist	#3/4	
		Dry		
Compaction Method	X	Mechanical Ram	#3/8	
		Manual Ram	#4	41.0

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3818	3900	3852			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	2010	2092	2044			
Wet Weight of Soil + Cont. (g)	796.2	1119.8	912.8			
Dry Weight of Soil + Cont. (g)	746.9	1027.8	822.2			
Weight of Container (g)	88.9	76.0	76.9			
Moisture Content (%)	7.49	9.67	12.16			
Wet Density (pcf)	133.5	138.9	135.7			
Dry Density (pcf)	124.2	126.7	121.0			

Maximum Dry Density (pcf) 126.8 **Optimum Moisture Content (%)** 9.5

Corrected Dry Density (pcf) 141.1 **Corrected Moisture Content (%)** 6.0

- [X] **Procedure A**
  Soil Passing No. 4 (4.75 mm) Sieve
  Mold : 4 in. (101.6 mm) diameter
  Layers : 5 (Five)
  Blows per layer : 25 (twenty-five)
  May be used if +#4 is 20% or less

- [ ] **Procedure B**
  Soil Passing 3/8 in. (9.5 mm) Sieve
  Mold : 4 in. (101.6 mm) diameter
  Layers : 5 (Five)
  Blows per layer : 25 (twenty-five)
  Use if +#4 is >20% and +3/8 in. is 20% or less

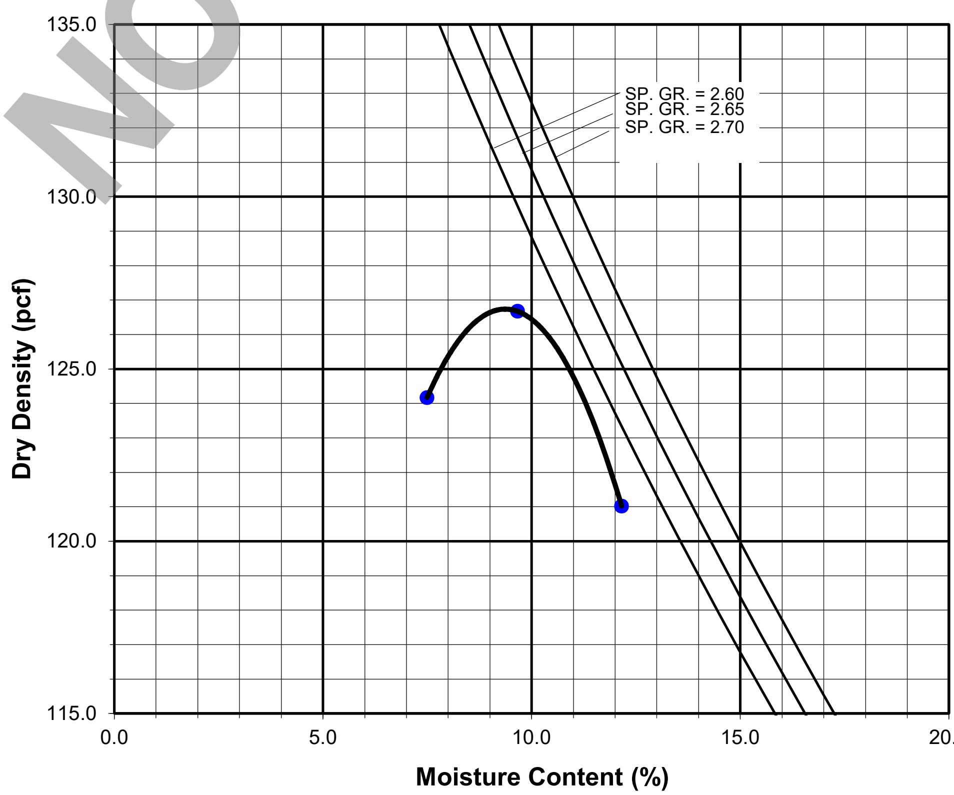
- [ ] **Procedure C**
  Soil Passing 3/4 in. (19.0 mm) Sieve
  Mold : 6 in. (152.4 mm) diameter
  Layers : 5 (Five)
  Blows per layer : 56 (fifty-six)
  Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Calico Ghost Town Bridge Replacement	Tested By:	ACS/GEB	Date:	01/05/24
Project No.:	38.0000020706	Checked By:	A. Santos	Date:	01/09/24
Boring No.:	LB-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Reddish brown clayey sand with gravel (SC)g				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0045
Wt. Comp. Soil + Mold	(g)	607.90	444.59
Wt. of Mold	(g)	187.70	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	835.60	632.29
Dry Wt. of Soil + Cont.	(g)	771.60	575.70
Wt. of Container	(g)	0.00	187.70
Moisture Content	(%)	8.29	14.59
Wet Density	(pcf)	126.8	133.5
Dry Density	(pcf)	117.0	116.5
Void Ratio		0.440	0.447
Total Porosity		0.306	0.309
Pore Volume	(cc)	63.3	64.2
Degree of Saturation (%) [S_{meas}]		**50.9**	88.1

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
01/05/24	12:27	1.0	0	0.4800
01/05/24	12:37	1.0	10	0.4800
Add Distilled Water to the Specimen				
01/05/24	13:00	1.0	23	0.4830
01/06/24	16:30	1.0	1673	0.4845
01/08/24	7:03	1.0	3986	0.4845

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**5**

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D6913

Leighton

Project Name:	Calico Ghost Town Bridge Replacement	Tested By:	ACS/KJ	Date:	01/02/24
Project No.:	38.0000020706	Checked By:	A. Santos	Date:	01/09/24
Boring No.:	LB-1	Depth (feet):	0-5		
Sample No.:	B-1				
Soil Identification:	Reddish brown silty sand with gravel (SM)g				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	P-7	903	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	6675.7	632.2	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	278.6	109.9	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	6397.1	522.3	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	903
	Wt. of Dry Soil + Container (g)	478.6
	Wt. of Container (g)	109.9
	Dry Wt. of Soil Retained on # 200 Sieve (g)	368.7

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5	0.0		100.0
1"	25.0	68.2		98.9
3/4"	19.0	264.2		95.9
1/2"	12.5	604.7		90.5
3/8"	9.5	842.3		86.8
#4	4.75	1523.5		76.2
#8	2.36		73.9	65.4
#16	1.18		146.7	54.8
#30	0.600		212.2	45.2
#50	0.300		278.2	35.6
#100	0.150		331.0	27.9
#200	0.075		365.7	22.8
PAN				

GRAVEL: **24 %**

SAND: **53 %**

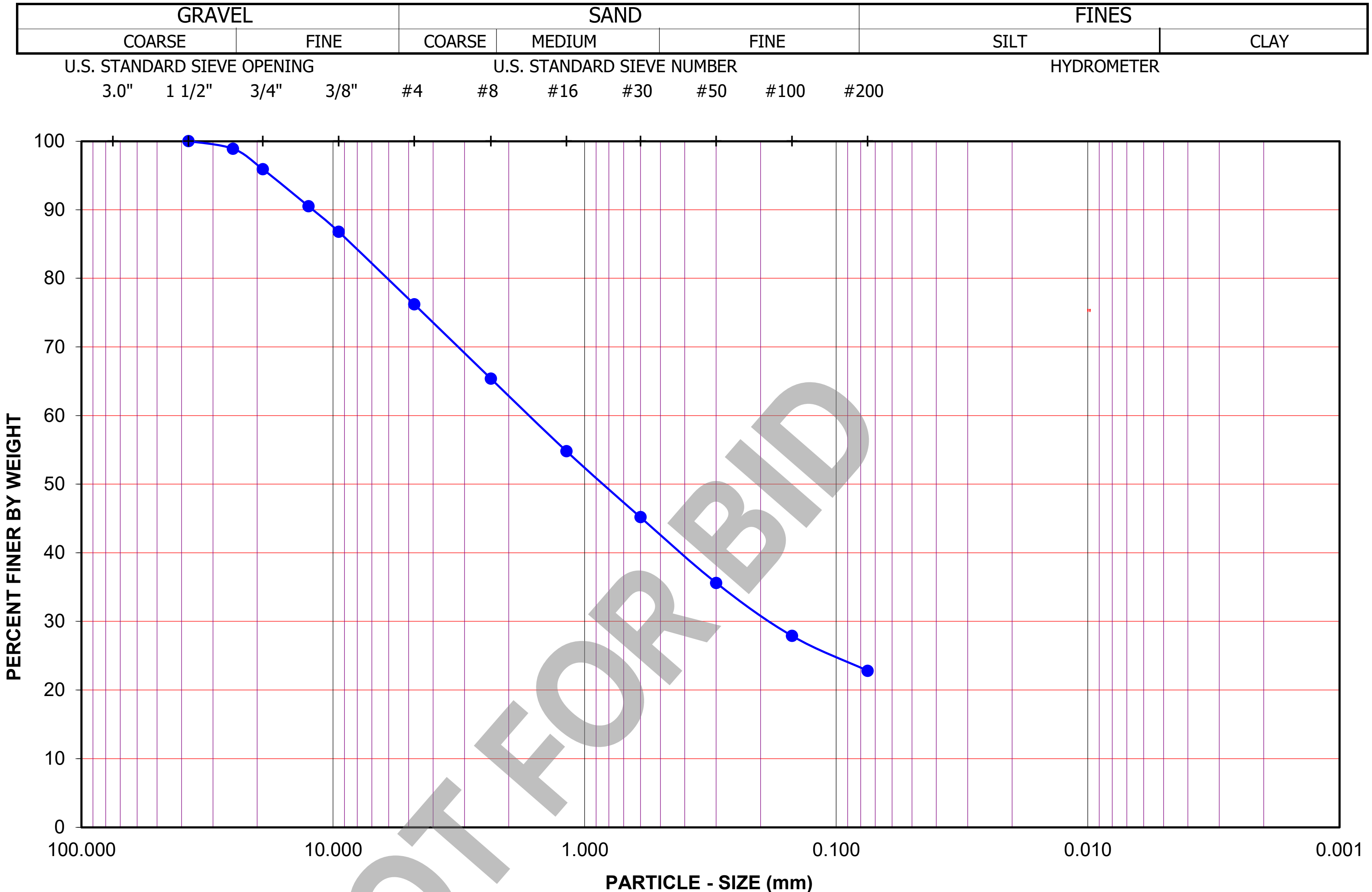
FINES: **23 %**

GROUP SYMBOL: **(SM)g**

$Cu = D60/D10 =$ ______

$Cc = (D30)^2/(D60*D10) =$ ______

Remarks:



Project Name: Calico Ghost Town Bridge Replacement

Project No.: 38.0000020706

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: LB-1

Sample No.: B-1

Depth (feet): 0-5

Soil Type : (SM)g

Soil Identification: Reddish brown silty sand with gravel (SM)g

GR:SA:FI : (%) 24 : 53 : 23

Jan-24

Boring No.	LB-2								
Sample No.	S-2								
Depth (ft.)	10								
Sample Type	SPT								
Soil Identification	Light brown silty sand with gravel (SM)g								
Moisture Correction									
Wet Weight of Soil + Container (g)	0.00								
Dry Weight of Soil + Container (g)	0.00								
Weight of Container (g)	1.00								
Moisture Content (%)	0.00								
Sample Dry Weight Determination									
Weight of Sample + Container (g)	613.80								
Weight of Container (g)	219.20								
Weight of Dry Sample (g)	394.60								
Container No.:									
After Wash									
Method (A or B)	A								
Dry Weight of Sample + Cont. (g)	489.70								
Weight of Container (g)	219.20								
Dry Weight of Sample (g)	270.50								
% Passing No. 200 Sieve	**31.4**								
% Retained No. 200 Sieve	68.6								

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

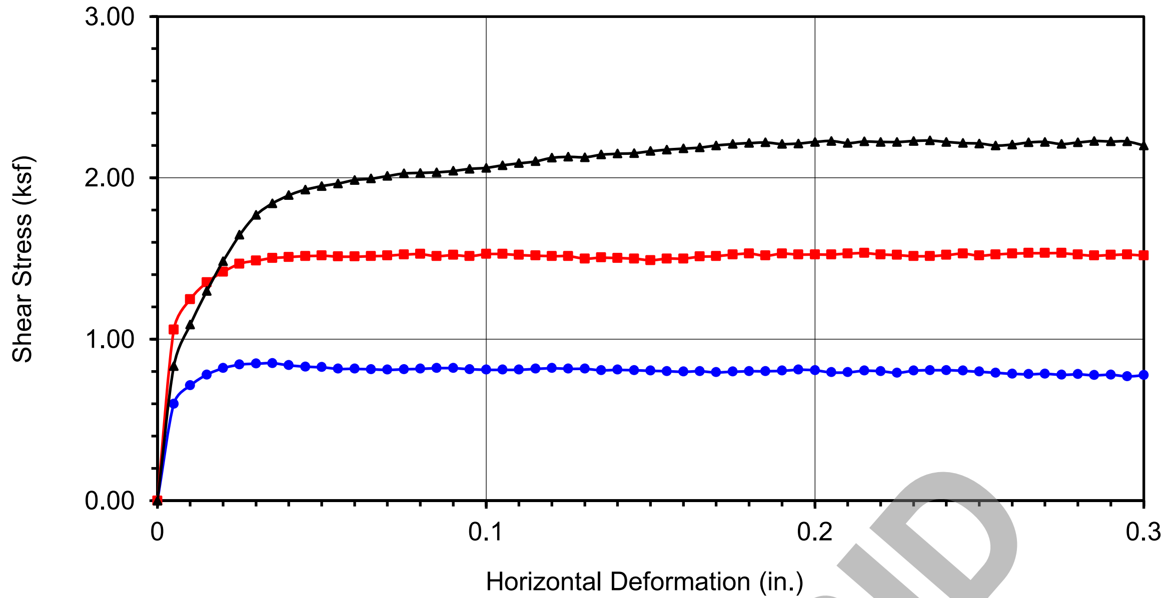
Project Name: Calico Ghost Town Bridge Replacement
Project No.: 38.0000020706
Tested By: ACS/KJ Date: 01/02/24

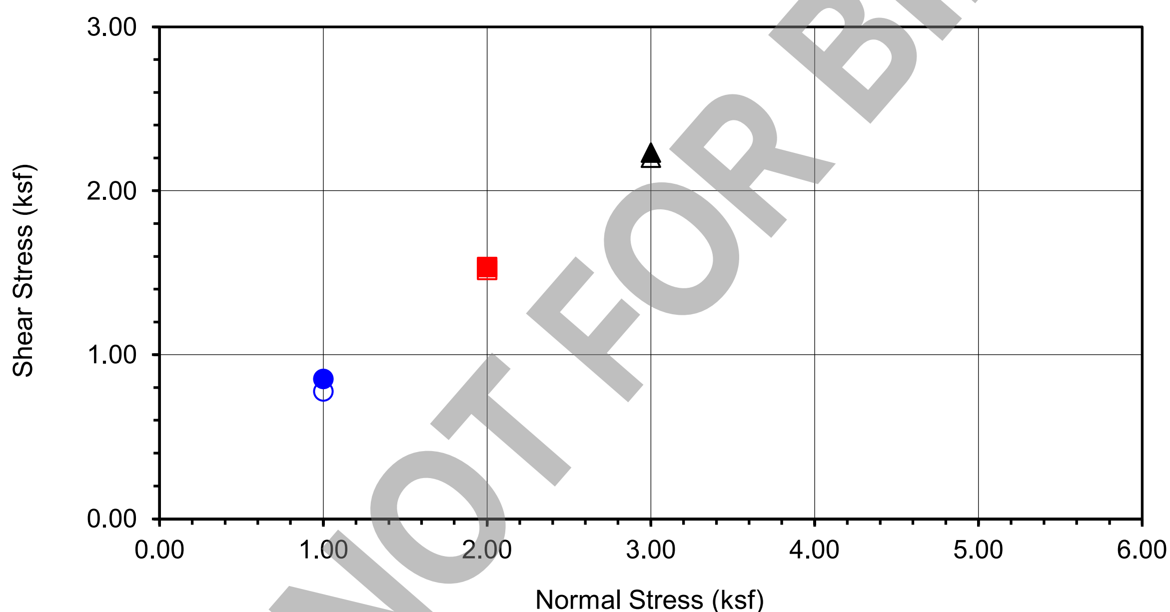
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Calico Ghost Town Bridge Replacement	Tested By:	G. Bathala	Date:	01/08/24
Project No.:	038.0000020706	Checked By:	A. Santos	Date:	01/10/24
Boring No.:	LB-2	Sample Type:	Bulk, 90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		
Soil Identification:	Reddish brown clayey sand with gravel (SC)g				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	193.20	196.22	196.34
Weight of Ring(gm):	42.55	45.49	45.26
Before Shearing			
Weight of Wet Sample+Cont.(gm):	156.49	156.49	156.49
Weight of Dry Sample+Cont.(gm):	147.55	147.55	147.55
Weight of Container(gm):	51.17	51.17	51.17
Vertical Rdg.(in): Initial	0.2521	0.2590	0.0000
Vertical Rdg.(in): Final	0.2589	0.2776	-0.0185
After Shearing			
Weight of Wet Sample+Cont.(gm):	216.20	216.57	205.93
Weight of Dry Sample+Cont.(gm):	196.77	197.74	187.48
Weight of Container(gm):	62.62	61.82	52.56
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

Bulk, 90% Remold

Soil Identification:
Reddish brown clayey sand with gravel (SC)g

Normal Stress (kip/ft²)	1.000	2.000	3.000
Peak Shear Stress (kip/ft²)	● 0.852	■ 1.534	▲ 2.232
Shear Stress @ End of Test (ksf)	○ 0.777	□ 1.518	△ 2.201
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.28	9.28	9.28
Dry Density (pcf)	114.7	114.7	115.0
Saturation (%)	53.3	53.4	53.7
Soil Height Before Shearing (in.)	0.9932	0.9814	0.9815
Final Moisture Content (%)	14.5	13.9	13.7

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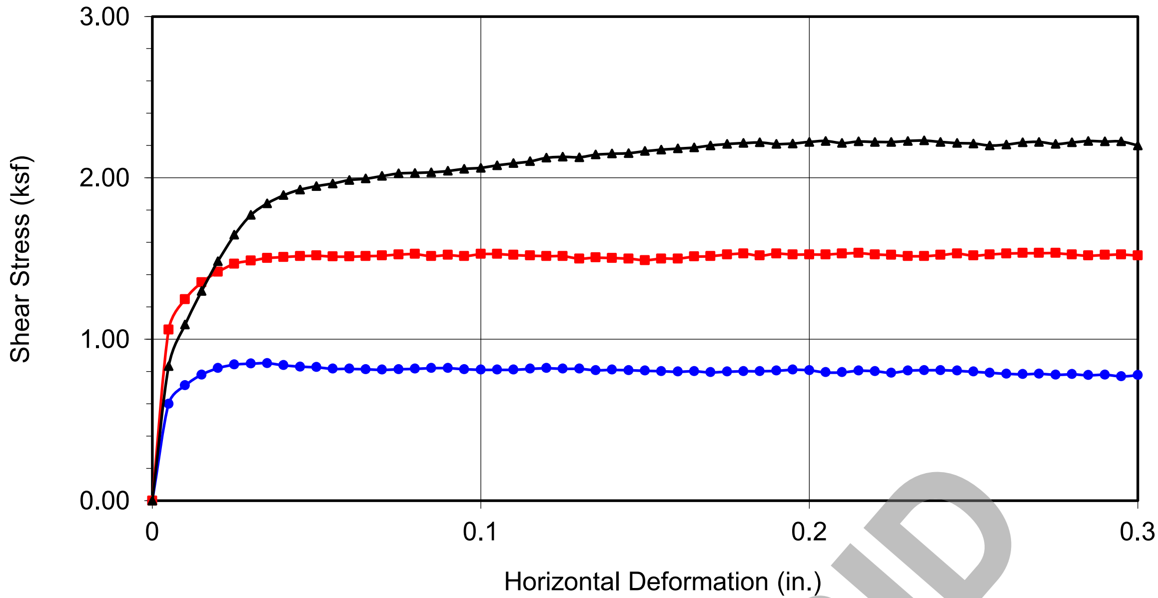
DIRECT SHEAR TEST RESULTS

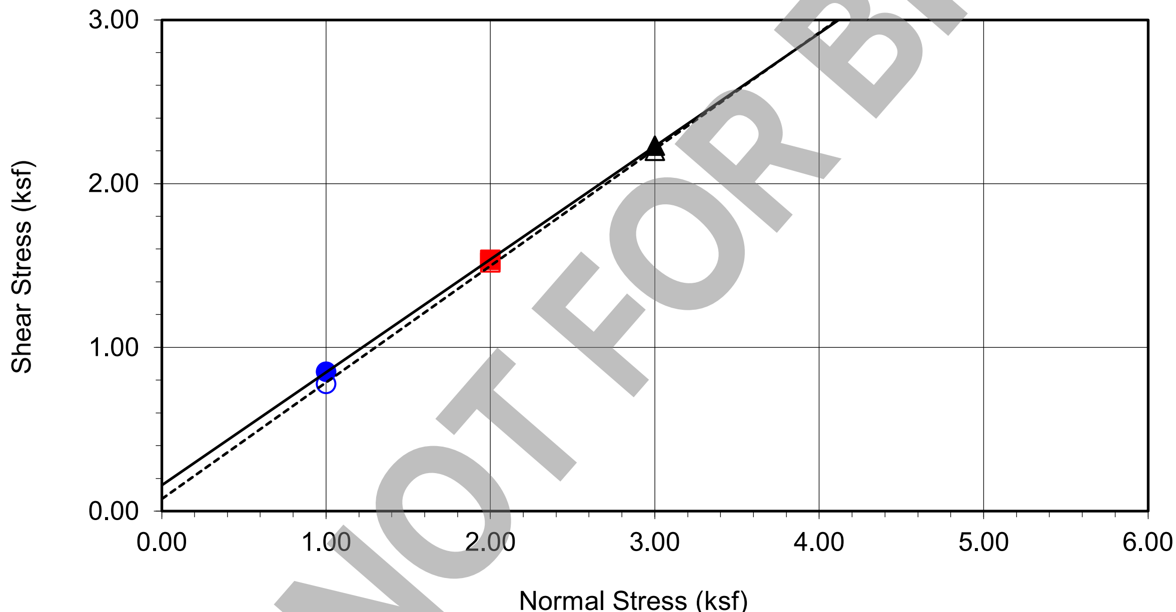
Consolidated Drained - ASTM D 3080

Project No.: 038.0000020706

Calico Ghost Town Bridge Replacement

01-24





Boring No.	**LB-2**
Sample No.	**B-1**
Depth (ft)	**0-5**
Sample Type: Bulk, 90% Remold	
Soil Identification: Reddish brown clayey sand with gravel (SC)g	

Strength Parameters

	C (psf)	ϕ (°)
Peak	159	35
Ultimate	75	35

Normal Stress (kip/ft²)	1.000	2.000	3.000
Peak Shear Stress (kip/ft²)	● 0.852	■ 1.534	▲ 2.232
Shear Stress @ End of Test (ksf)	○ 0.777	□ 1.518	△ 2.201
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.28	9.28	9.28
Dry Density (pcf)	114.7	114.7	115.0
Saturation (%)	53.3	53.4	53.7
Soil Height Before Shearing (in.)	0.9932	0.9814	0.9815
Final Moisture Content (%)	14.5	13.9	13.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 038.0000020706

Calico Ghost Town Bridge Replacement

01-24

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Calico Ghost Town Bridge Replacement Tested By : ACS/GEB Date: 01/03/24

Project No. : 038.0000020706 Checked By: A. Santos Date: 01/10/24

Boring No.	LB-1			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Reddish brown (SM)g			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.60			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	8			
Crucible No.	010			
Furnace Temperature (°C)	860			
Time In / Time Out	10:00/10:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	15.9244			
Wt. of Crucible (g)	15.9191			
Wt. of Residue (g) (A)	0.0053			
PPM of Sulfate (A) x 41150	218.10			
PPM of Sulfate, Dry Weight Basis	**218**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	5			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	300			
PPM of Chloride, Dry Wt. Basis	**300**			

pH TEST, DOT California Test 643

pH Value	**9.71**			
Temperature °C	20.1			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Calico Ghost Town Bridge Replacement	Tested By :	J. Domingo Date: 01/09/24
Project No. :	038.0000020706	Checked By:	A. Santos Date: 01/10/24
Boring No.:	LB-1	Depth (ft.) :	0-5
Sample No. :	B-1		

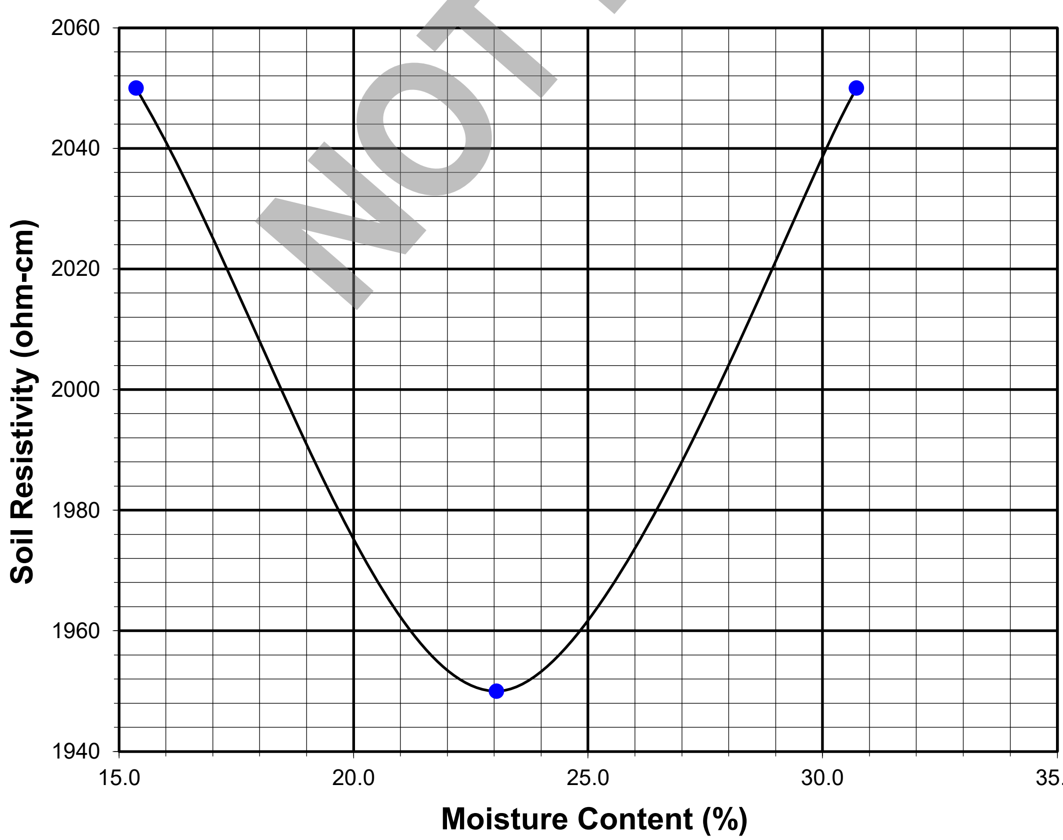
Soil Identification:* Reddish brown (SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.36	2050	2050
2	30	23.04	1950	1950
3	40	30.72	2050	2050
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.19
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1950	**23.0**	**218**	**300**	**9.71**	**20.1**



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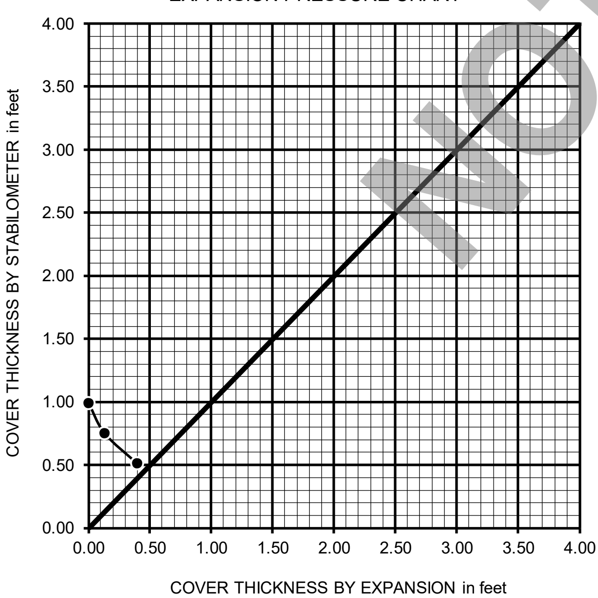
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Calico Ghost Town Bridge Replacement	PROJECT NUMBER:	038.0000020706
BORING NUMBER:	LB-2	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Reddish brown clayey sand with gravel (SC)g	DATE COMPLETED:	1/4/2024

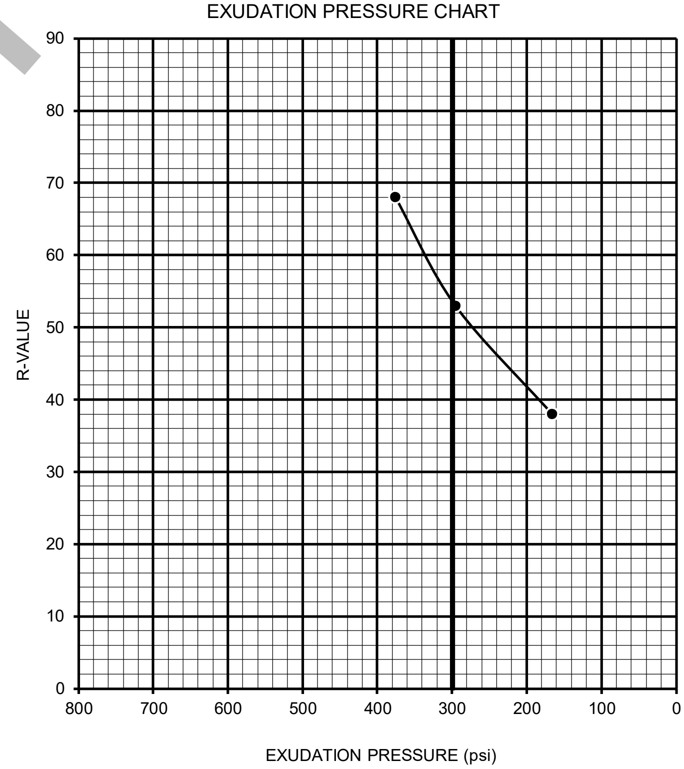
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	9.6	10.7	11.7
HEIGHT OF SAMPLE, Inches	2.51	2.48	2.50
DRY DENSITY, pcf	127.3	126.4	125.9
COMPACTOR PRESSURE, psi	120	70	50
EXUDATION PRESSURE, psi	376	295	166
EXPANSION, Inches x 10exp-4	12	4	0
STABILITY Ph 2,000 lbs (160 psi)	32	48	68
TURNS DISPLACEMENT	4.50	5.10	5.60
R-VALUE UNCORRECTED	69	53	38
R-VALUE CORRECTED	68	53	38

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.51	0.75	0.99
EXPANSION PRESSURE THICKNESS, ft.	0.40	0.13	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	69
R-VALUE BY EXUDATION:	53
EQUILIBRIUM R-VALUE:	53

APPENDIX D

SEISMIC

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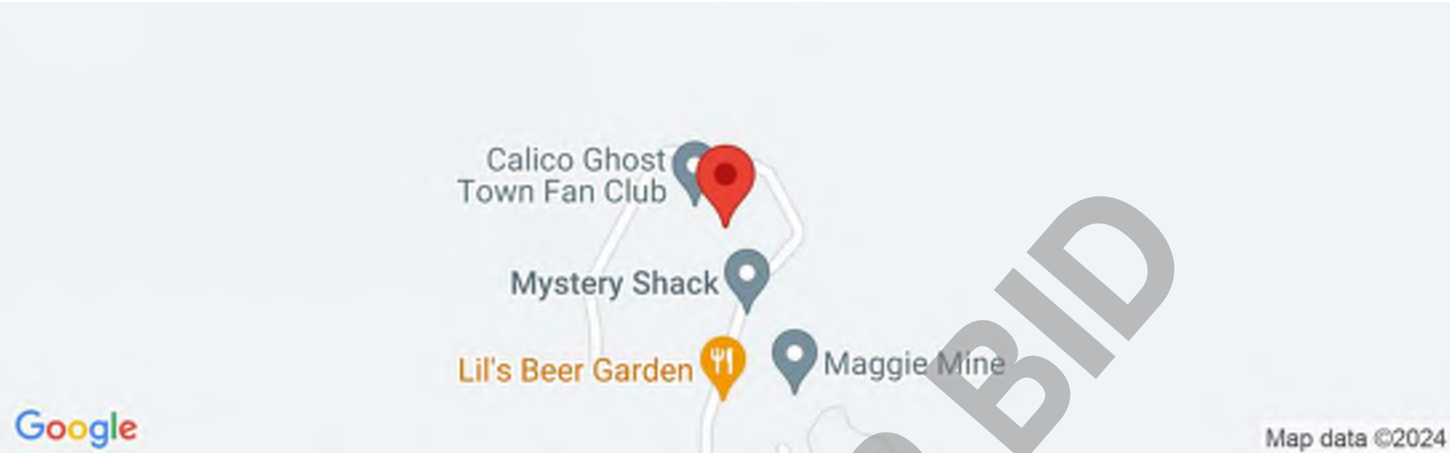
a verdantas company

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

OSHPD

Latitude, Longitude: 34.9508, -116.8649



Date	1/12/2024, 12:03:59 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	B - Rock

Type	Value	Description
S_S	1.769	MCE_R ground motion. (for 0.2 second period)
S_1	0.612	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.592	Site-modified spectral acceleration value
S_{M1}	0.49	Site-modified spectral acceleration value
S_{DS}	1.061	Numeric seismic design value at 0.2 second SA
S_{D1}	0.326	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	0.9	Site amplification factor at 0.2 second
F_v	0.8	Site amplification factor at 1.0 second
PGA	0.794	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.715	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.769	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.997	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.325	Factored deterministic acceleration value. (0.2 second)
S1RT	0.612	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.692	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.882	Factored deterministic acceleration value. (1.0 second)
PGAd	0.972	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.794	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.886	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.884	Mapped value of the risk coefficient at a period of 1 s
C_V	0.9	Vertical coefficient

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Latitude

Decimal degrees

34.9508

Longitude

Decimal degrees, negative values for western longitudes

-116.8649

Site Class

1150 m/s (Site class B)

Spectral Period

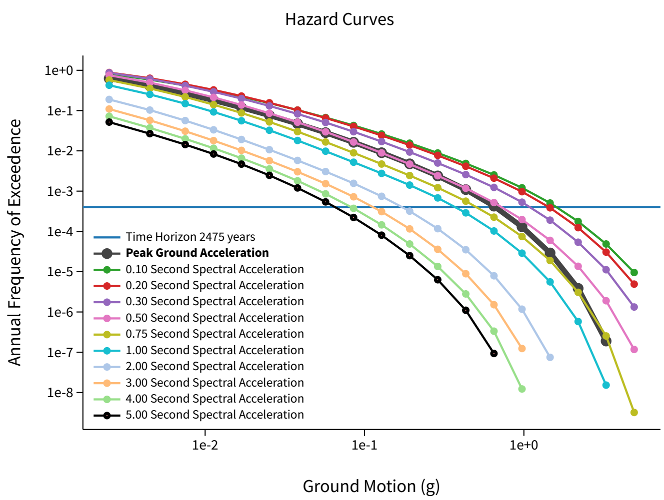
Peak Ground Acceleration

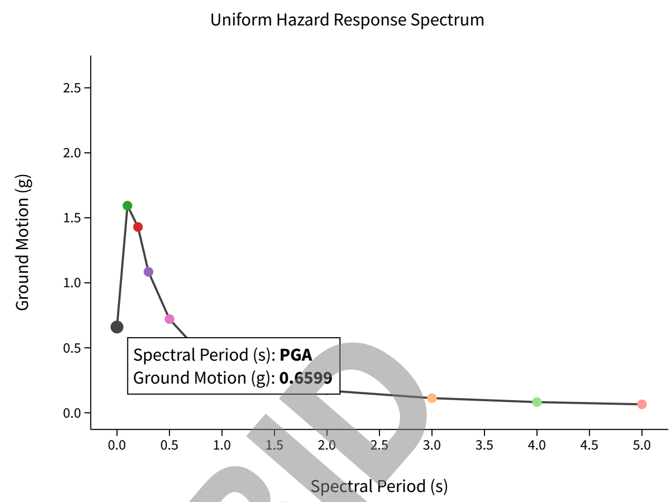
Time Horizon

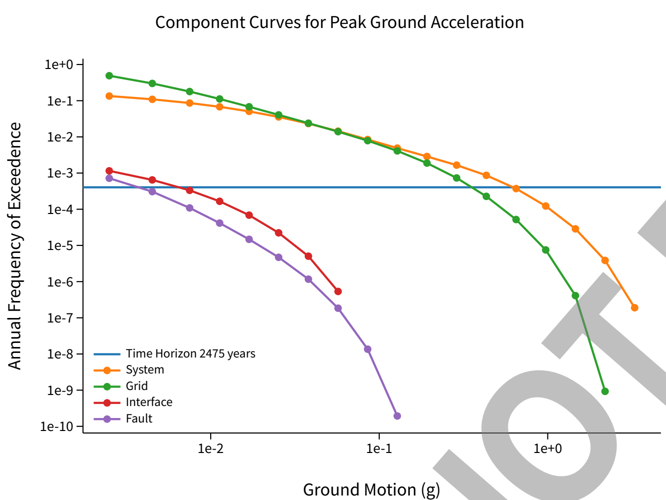
Return period in years

2475

Hazard Curve





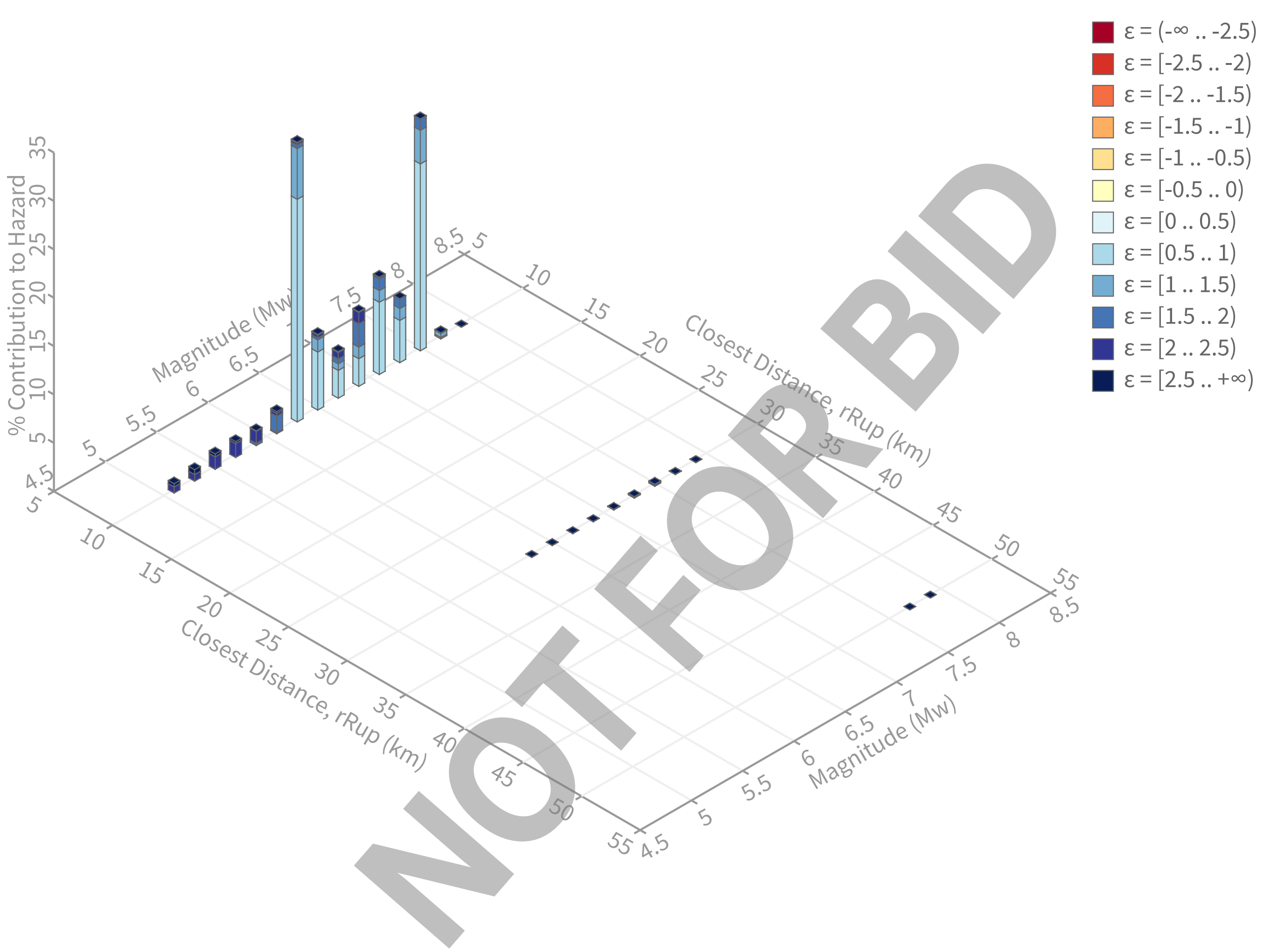


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.65989046 g

Recovered targets

Return period: 2738.4143 yrs
Exceedance rate: 0.00036517484 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.01 %

Mean (over all sources)

m: 6.77
r: 3.61 km
ε_0: 1.08 σ

Mode (largest m-r bin)

m: 6.28
r: 1.79 km
ε_0: 0.96 σ
Contribution: 28.88 %

Mode (largest m-r-ε_0 bin)

m: 6.28
r: 1.56 km
ε_0: 0.89 σ
Contribution: 23 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							46.10
Calico-Hidalgo [16]		1.56	6.79	0.81	116.865°W	34.957°N	359.05	36.37
Calico-Hidalgo [15]		5.40	7.38	1.10	116.813°W	34.931°N	115.55	4.05
Gravel Hills-Harper Lk [0]		10.06	7.27	1.66	116.919°W	34.872°N	209.31	1.99
UC33brAvg_FM32	System							44.49
Calico-Hidalgo [16]		1.56	6.82	0.80	116.865°W	34.957°N	359.05	34.84
Calico-Hidalgo [15]		5.40	7.37	1.10	116.813°W	34.931°N	115.55	4.01
Gravel Hills-Harper Lk [0]		10.06	7.27	1.66	116.919°W	34.872°N	209.31	2.06
UC33brAvg_FM31 (opt)	Grid							4.71
PointSourceFinite: -116.865, 35.009		8.11	5.67	2.13	116.865°W	35.009°N	0.00	1.78
PointSourceFinite: -116.865, 35.009		8.11	5.67	2.13	116.865°W	35.009°N	0.00	1.76
UC33brAvg_FM32 (opt)	Grid							4.71
PointSourceFinite: -116.865, 35.009		8.11	5.67	2.13	116.865°W	35.009°N	0.00	1.78
PointSourceFinite: -116.865, 35.009		8.11	5.67	2.13	116.865°W	35.009°N	0.00	1.76

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: Calico Ghost Town Bridge; Case 1; PGAm 0.715; design GW 300; No overex 0

Project No.: 20706

Jan 2024

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	400	300		0	2331	2031
LB-2	400	300		0	2327	2027
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)

General Parameters:	
a_{max} =	0.72g
M_W =	6.3
MSF eq:	1
MSF =	1.56
Hammer Efficiency =	84
C_E =	1.40
C_B =	1
C_S for SPT?	TRUE
Unlined, but room for liner	
Rod Stickup (feet) =	3
Ring sample correction =	0.65

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Calico Ghost Town Bridge; Case 1; PGAm 0.715; design GW 300; No overex 0

Project No.: 20706

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	Cs	N_m (corrected for Cs and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	**$CRR_{7.5}$**	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	**Liquefaction Factor of Safety**	$(N_1)_{60CS}$ (for Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-1	0 to 3.8	2.5	3.8		20	120	100	1	1.3	130.0	300	232.1	254.1	>Range	300	0.46	0.30	**NonLiq**	254.1	0.00		0.00	0.0
LB-1	3.8 to 6.3	5	2.5		60	120	100	1	1.3	130.0	600	232.1	283.5	>Range	600	0.46	0.29	**NonLiq**	283.5	0.00		0.00	0.0
LB-1	6.3 to 8.8	7.5	2.5		20	120	100	1	1.3	130.0	900	221.8	243.0	>Range	900	0.46	0.29	**NonLiq**	243.0	0.01		0.00	0.0
LB-1	8.8 to 12.5	10	3.8		30	120	100	1	1.3	130.0	1200	204.1	240.3	>Range	1200	0.45	0.29	**NonLiq**	240.3	0.01		0.00	0.0
LB-1	12.5 to 17.5	15	5.0		70	120	100	1	1.3	130.0	1800	166.6	204.9	>Range	1800	0.45	0.29	**NonLiq**	204.9	0.01		0.00	0.0
LB-1	17.5 to 22.5	20	5.0		70	120	100	1	1.3	130.0	2400	161.3	198.5	>Range	2400	0.44	0.28	**NonLiq**	198.5	0.01		0.01	0.0
LB-1	22.5 to 27.5	25	5.0		70	120	100	1	1.3	130.0	3000	144.2	178.1	>Range	3000	0.44	0.28	**NonLiq**	178.1	0.01		0.01	0.0
LB-1	27.5 to 32.0	30	4.5		70	120	100	1	1.3	130.0	3600	138.6	171.3	>Range	3600	0.43	0.28	**NonLiq**	171.3	0.01		0.01	0.0
LB-2	0 to 3.8	2.5	3.8		40	120	100	2	1	65.0	300	116.0	144.2	>Range	300	0.46	0.30	**NonLiq**	144.2	0.00		0.00	0.0
LB-2	3.8 to 6.3	5	2.5		60	120	100	1	1.3	130.0	600	232.1	283.5	>Range	600	0.46	0.29	**NonLiq**	283.5	0.00		0.00	0.0
LB-2	6.3 to 8.8	7.5	2.5		25	120	100	2	1	65.0	900	110.9	127.9	>Range	900	0.46	0.29	**NonLiq**	127.9	0.01		0.00	0.0
LB-2	8.8 to 12.5	10	3.8		31	120	100	1	1.3	130.0	1200	204.1	242.0	>Range	1200	0.45	0.29	**NonLiq**	242.0	0.01		0.00	0.0
LB-2	12.5 to 17.5	15	5.0		25	120	100	2	1	65.0	1800	83.3	97.2	>Range	1800	0.45	0.29	**NonLiq**	97.2	0.01		0.01	0.0
LB-2	17.5 to 22.0	20	4.5		20	120	100	1	1.3	130.0	2400	161.3	177.7	>Range	2400	0.44	0.28	**NonLiq**	177.7	0.01		0.01	0.0

APPENDIX E

GENERAL EARTHWORK AND
GRADING GUIDE SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

APPENDIX F

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled*. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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