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GEOTECHNICAL EXPLORATION

MOJAVE NARROWS REGIONAL PARK ACCESSIBLE CAMPSITE RESTROOM

18000 YATES ROAD

VICTORVILLE AREA, UNINCORPORATED SAN BERNARDINO COUNTY, CALIFORNIA

Prepared For

SAN BERNARDINO COUNTY PROJECT AND FACILITIES MANAGEMENT DEPARTMENT
385 NORTH ARROWHEAD AVENUE, THIRD FLOOR
SAN BERNARDINO, CA 92415-0180

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Project Number 12099.006

August 3, 2023

Leighton Consulting, Inc.
A Leighton Group Company

August 3, 2023

Project No. 12099.006

San Bernardino County Project and Facilities Management Department
385 North Arrowhead Avenue, Third Floor
San Bernardino, California 92415

Attention: Mr. Ryan Johnson
Project Manager III – Project and Facilities Management Department

Subject: Geotechnical Exploration
Mojave Narrows Regional Park Accessible Campsite Restroom
18000 Yates Road
Victorville Area, Unincorporated San Bernardino County, California

In accordance with your authorization, Leighton Consulting, Inc. (Leighton) has conducted this geotechnical exploration for use in designing the proposed restroom/shower building and ADA parking to be constructed at the existing Mojave Narrows Regional Park along Horseshoe Lane, south of Horseshoe Lake, in the Victorville area of unincorporated San Bernardino County, California. The purpose of this study has been to collect subsurface geotechnical data at the site, evaluate the proposed improvements with respect to the site geotechnical conditions, and provide geotechnical recommendations for design and construction.

Based on this geotechnical exploration, construction of the proposed restroom/shower building is feasible from a geotechnical standpoint. The most significant geotechnical issues for this project are those related to the potential for strong seismic shaking, presence of liquefiable soils, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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1.0 INTRODUCTION

1.1 Site Location and Description

The project site is located within the existing Mojave Narrows Regional Park (located at 18000 Yates Road), south of Horseshoe Lake, southwest of Horseshoe Lane in the Victorville area of unincorporated San Bernardino County, California. The approximately 886-acre park is bordered by the Santa Fe Railroad to the west, Yates Road and the Spring Valley Lake area to the south, residential housing in the Desert Knolls area to the east, and by the SR-18 Freeway to the north (see Figure 1, *Site Location Map*). The proposed improvements cover an area of approximately 4,500 square feet and are located directly southwest of Horseshoe Lane (approximate GPS coordinates 34.5101, -117.2762). The park is currently open to the public.

Based on elevation data from topographic maps, Google Earth, and our field observations, the site is relatively flat and generally drains to the north. The existing ground elevation at the proposed improvement area is approximately 2,760 feet above mean sea level (msl) based on Google Earth's elevation model. Grading plans show that existing topography within the project area has less than 1 foot in elevation variation.

1.2 Proposed Development

Our understanding of the project is based on correspondence with you, the building plans *titled San Bernardino County, Mojave Narrows Regional Park Accessible Campsite, Project #10.10.4402* (Sheets T1/1, A1.1, A1.2, C-1, and C-4 only) prepared by STK Architecture, Inc., plotted February 22, 2023, and the shop drawings (33 pages) prepared by CXT Precast Products approved on February 12, 2021. We understand that the San Bernardino County Project and Facilities Management Department is proposing to construct a prefabricated restroom/shower building and accommodating ADA parking lot with an accessible path of travel. The proposed project will be located within the existing Mojave Narrows Regional Park, south of Horseshoe Lake, and southwest of Horseshoe Lane. The proposed restroom/shower building will have an approximate footprint of 340 Square feet (SF) and will be located south of the proposed parking area which will be composed of four parking spots. The preliminary plans show that proposed grade changes will generally include less than 3 feet of fill relative to existing grades. A 4-inch diameter sewer lateral will be constructed to connect the proposed restroom to the existing manhole to the southwest.

The building plans indicate another ADA parking area across Horseshoe Lane to the northeast of the proposed restroom. Upon arrival on site for our field exploration, it was observed that this ADA parking area had already been constructed. This parking area northeast of Horseshoe Lane is not a part of the scope of work for this report.

1.3 Purpose of Investigation

The purpose of this study has been to evaluate the geotechnical conditions of the project site with respect to the proposed improvements and to provide geotechnical recommendations for design and construction.

1.4 Scope of Investigation

Our geotechnical exploration included hollow-stem auger soil borings, laboratory testing, and geotechnical analysis to evaluate existing geotechnical conditions and to develop the conclusions and recommendations contained in this report. The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical and geologic maps and reports and aerial photographs available from our in-house library, available online, or those provided by you.

- Utility Coordination: We contacted Dig Alert (811) prior to excavating borings so that utility companies could mark utilities onsite. We coordinated our work with you and a site representative.

- Field Exploration: A total of two (2) hollow-stem auger borings were logged and sampled onsite to evaluate subsurface conditions. One geotechnical boring was drilled within the proposed building structure footprint, and the other within the proposed ADA parking area. These borings were drilled by a subcontracted rig to depths ranging from approximately 16.5 to 51.5 feet below the existing ground surface (bgs). Encountered earth materials were logged by our field representative and described in accordance with the Unified Soil Classification System (USCS). Representative bulk soil samples were collected from the borings at shallow depths, within the upper 5 feet. Relatively undisturbed soil samples were obtained at select interval depths within these borings using a Modified California ring-lined sampler. An unlined, 2-inch outside diameter Standard Penetration Test (SPT) split-spoon sampler was also used in collecting samples. Both generally followed respective ASTM D3550 and ASTM D1586 sampling procedures. Sampling resistance blow counts were obtained by dropping a 140-pound automatic hammer through a 30-inch free fall onto a sampling rod anvil. The number of blows was recorded for each 6 inches of penetration (ASTM D1586).

Borings were backfilled approximately to the level of the existing surface with spoils and cuttings generated during drilling. Logs of the borings are presented in Appendix B. Approximate borehole locations are shown on the accompanying Figure 2, *Exploration Location Map*.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation included:

  - Maximum dry density and optimum moisture content
  - In situ moisture content and dry density
  - Grain-size distribution
  - Atterberg Limits
  - R-value
  - Expansion Index
  - Swell/Collapse Potential
  - Direct Shear
  - Water-soluble sulfate concentration in the soil
  - Resistivity, chloride content and pH

  In situ moisture content and dry density are provided on the boring logs. Remaining test results are provided in Appendix C, *Laboratory Test Results*.

- Engineering Analysis: Data obtained from our background review, along with data from our field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.

- Report Preparation: Results of our geotechnical exploration have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for design and construction of the proposed development.

2.0 FINDINGS

2.1 Regional Geology

The site is located in the western Mojave Desert, in San Bernardino County California, and is part of the Mojave Desert geomorphic province, a broad interior region of isolated mountain ranges separated by broad desert plains and deep alluvial valleys. The Mojave province is wedged between the Garlock Fault (southern boundary of the Sierra Nevada) and the San Andreas Fault, where it bends northerly from its northwest trend. The northern boundary of the Mojave province is separated from the prominent Basin and Range by the eastern extension of the Garlock Fault.

The project site is located along the margin of the floodplain of the Mojave River. The Mojave River was a drainage system that had flowed towards the west until the late Tertiary, when uplift of the Traverse Ranges Mountains occurred to the south of the current Mojave Desert. This uplift changed the course of the drainages to flow towards the north and east, filling the basins and overflowing the divides until the current course of the Mojave River towards Soda Lake was established.

The project site has been regionally mapped (Dibblee and Minch, 2008) to be underlain by alluvial silt, sand, and gravel of valley areas derived from higher ground (Qa) and sands of the Mojave River channel (Qg).

The site is located approximately 7.9 miles northwest of the western segment of the North Frontal thrust system; 11.4 miles southwest of the Helendale-South Lockhart fault zone; 15.3 miles north of the Cleghorn fault zone; and 19.5 miles northeast of the San Andreas fault zone (see Figure 4, *Regional Fault and Historical Seismicity Map*).

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by native alluvial deposits. During our exploration, we did not differentiate between the two regionally mapped alluvial deposits. Undocumented fill was not encountered in our soil borings, which reached total depths ranging of approximately 16.5 and 51.5 feet below the ground surface (bgs).

The alluvial soil encountered within our excavations generally consisted of silty to clayey sands and sands with silt in the upper 35 feet and sandy lean clay from 35 feet to 51.5 feet bgs. These native soils were firm and loose in the upper 18 feet bgs and generally became denser below a depth of about 20 feet bgs. The upper soils were observed to be slightly moist to moist, and were saturated at depths below 9 feet where groundwater was encountered. Collected samples within the upper approximately 10 feet were tested to have moisture contents ranging from 11 to 33 percent and dry densities ranging from 86 pcf to 109 pcf. More detailed descriptions of the subsurface soil are presented on the boring logs in Appendix B.

Based on review of the Geologic Map of the Shadow Mountains and Victorville Quadrangles Map DF-387 (Dibblee, 2008), the site is mapped as having two main map symbols within the proposed project site. The unit names encountered at the site consist of sand of the Mojave River channel (Qg) and surficial sediments consisting of alluvial silt, sand and gravel derived from adjacent higher grounds (Qa).

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on this study and the near-surface encountered loose sands, the upper portion of native soils are considered compressible. Removal/recompaction of near surface alluvium is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. An undisturbed sample at a depth of 20 feet was tested for collapse potential per ASTM D 4546. Based on the results, soil collapse and consolidation are not significant issues with development. However, the field Standard Penetration Test (SPT) blowcounts for soils within the upper 15 feet showed a loose consistency and groundwater levels are relatively shallow, therefore there is a potential for soil collapse. Removal/recompaction of near surface alluvium is recommended to reduce the potential for soil collapse.

2.2.2 Expansive Soils

Expansive soil contains significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on expansive soil are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of building foundations and slabs-on-grade could result.

An expansion index (EI) test performed on a shallow bulk sample yielded a measured EI of 28, which is classified as "low" expansion. Based on the encountered near-surface soils and laboratory test results, the onsite soils are anticipated to have "low" expansion potential.

2.2.3 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 American Concrete Institute (ACI) provisions, adopted by the 2022 CBC (CBC, 2022, Chapter 19, and ACI 318, 2014).

A representative near-surface soil sample was tested for soluble sulfate during this investigation. This test resulted in sulfate contents of less than 0.1 percent by weight, indicating negligible sulfate exposure (Exposure Class S0). Recommendations for concrete in contact with the soil are provided in Section 3.6

2.2.4 Resistivity, Chloride and pH

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

As a screening for potentially corrosive soil, a representative soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 650 ohm-cm, chloride content of 300 ppm, and pH of 7.4. Based on these results, the onsite soil is considered to be severely corrosive to metals. It is recommended that any buried pipe be made of non-ferrous material, or that any ferrous pipe be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if metallic utilities are planned.

2.3 Groundwater

Groundwater was encountered in both of our borings drilled on April 20th, 2023, at a depth of approximately 9 feet bgs. Horseshoe Lake is located approximately 130 feet northeast of our closest boring (LB-2). Groundwater data from State Well number 345128N117269W001, located approximately half a mile northeast from the vicinity of the proposed improvements, (CDWR, 2023a) indicated the shallowest groundwater measured was at an elevation of 2,751 feet above mean sea level from readings between the periods of 1996 and 2000, which correlates to a depth of approximately 9 feet bgs from existing grade at the proposed improvement area. The readings from this nearby well coincide with our encountered groundwater level.

Groundwater at the current encountered elevation is not anticipated to be a constraint for earthwork operations. However, the encountered groundwater could be a constraint for planned utilities if installation depths are near the encountered levels.

2.4 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region include surface rupture along active faults and strong ground shaking. The potential for fault rupture and seismic shaking are discussed below.

2.4.1 <u>Faulting</u>

The project site is located outside of a State or County designated Earthquake Fault Zone. Our review of available in-house literature indicates that there are no known active faults traversing or trending towards the site. The closest known active or potentially active faults are I the western segment of the North Frontal thrust system (approximately 7.9 miles northwest of the site); the Helendale-South Lockhart fault zone (approximately 11.4 miles southwest of the site); the Cleghorn fault zone (approximately 15.3 miles north of the site); and the San Andreas fault zone (approximately 19.5 miles northeast of the site).

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along these and other major active or potentially active faults near the project (see Figure 4, *Regional Fault and Historical Seismicity Map*).

2.4.2 Ground Shaking

The site has and will experience strong ground shaking during the life of the project 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.

Site Class D was selected based on evaluation of the average field Standard Penetration Resistance in accordance with field Standard Penetration Test blowcount data in accordance with ASCE 7-16 20.3 and 20.4. A summary of our site class calculations is included in Appendix D.

The following seismic parameters should be considered for design under the 2022 edition of the California Building Code (CBC). The following table lists seismic design parameters based on the 2022 CBC and ASCE 7-16 methodology:

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude: 34.5101, -117.2762	
Site Class Definition (1613A.2.2, ASCE 7-16 Ch 20)	D**
Mapped Spectral Response Acceleration at 0.2s Period (1613A.2.1), S_s	1.148 g
Mapped Spectral Response Acceleration at 1s Period (1613A.2.1), S_1	0.442 g
Short Period Site Coefficient at 0.2s Period (T1613A.2.3(1)), F_a	*1.041*
Long Period Site Coefficient at 1s Period (T1613A.2.3(2)), F_v	*1.858**
Adjusted Spectral Response Acceleration at 0.2s Period (1613A.2.3), S_{MS}	1.195 g
Adjusted Spectral Response Acceleration at 1s Period (1613A.2.3), S_{M1}	0.821* g
Design Spectral Response Acceleration at 0.2s Period (1613A.2.4), S_{DS}	0.796 g
Design Spectral Response Acceleration at 1s Period (1613A.2.4), S_{D1}	0.547* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), *PGA*	0.493 g
Site Coefficient for Mapped MCE_G *PGA* (11.8.3.2), F_{PGA}	1.107
Peak Ground Acceleration, mod w/ site effects (1803A.5.12; 11.8.3.2), PGA_M	0.546 g

* See Section 11.4.8 of ASCE 7-16. A site-specific ground motion hazard analysis in accordance with Section 21.2 of ASCE 7-16 is required for this site. **Per Supplement 3 to ASCE 7-16, a site-specific ground motion hazard analysis is not required where the value of the parameters S_{M1} and S_{D1} in the table are increased by 50%.**

**Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate the predominant modal earthquake has a magnitude of approximately 7.9 (MW) at a distance of approximately 31.4 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landslides, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential and Lateral Spreading

Liquefaction is the loss of soil strength or 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 this site for liquefaction hazards. The San Bernardino County Geologic Hazard Overlay Map EHFH C indicates this site to be mapped outside a zone of liquefaction susceptibility (San Bernardino County, 2010). The groundwater data from a well near the site and conditions encountered in our borings indicated a historically highest groundwater depth of approximately 9 feet bgs.

We have performed liquefaction analysis based on our geotechnical borings drilled at this site. Our liquefaction evaluation was based on:

- **Ground Motion**: A peak horizontal ground acceleration of 0.55g and a Moment Magnitude (Mw) of 7.9 was used.
- **Groundwater Depth**: As is customary in California, we have used a historically high groundwater table of 9 feet below ground surface.
- **Soil Classification and Density**: Soil classification and density was based solely on California ring-lined and Standard Penetration Test (SPT) drive samples, typically obtained at 5-foot depth intervals in our borings. Site soils were predominantly loose to very dense, coarse-grained material down to a depth of 30 feet bgs and stiff to hard, fine-grained material from 30 feet to 50 feet bgs.

Our analysis presented in Appendix D, *Seismic Hazard Analysis,* identifies potentially liquefiable soil layer at depths between 9 and 18 feet. The liquefiable layer consists of sand. The thickness of this potentially liquefiable layer has been probably overestimated due to the sampling interval of every 5 feet in depth. As much as 2.3 inches of liquefaction-induced settlement was estimated within this layer between a depth of 9 and 18 feet below existing grade. Proposed grades at the proposed building location are approximately 2 feet above existing grade.

Due to the relatively shallow liquefiable soils, there is a potential for lateral spreading. In order to reduce the potential for lateral spreading to tolerances that meet project requirements, we have included recommendations to implement a geogrid-reinforced granular mat at the building pad overexcavation depth.

2.5.2 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 9 feet below ground surface were used in the analysis. Based on our analysis, a potential for approximately 2.5 inches of total seismic settlement is estimated at the site. Results of our seismic settlement analysis is presented in Appendix D.

If the potential differential settlement is estimated as half of the total seismic settlement over a horizontal distance of 20 feet, this would result in a maximum 1.2 inches of differential settlement in 20 feet, or angular distortion of 0.005L. This would be within the differential settlement threshold of 0.0075L for “single-story structures with concrete or masonry wall systems” of Risk Category II, and threshold of 0.015L of “other single-story structures” as listed in Table 12.13-3 of ASCE 7-16. The structural engineer should determine Structure Type and Risk Category and evaluate whether the differential settlement estimates described above are tolerable. A copy of ASCE 7-16 Table 12.13-3 is provided as follows for reference.

Table 12.13-3 Differential Settlement Threshold

Structure Type	Risk Category		
	I or II	III	IV
Single-story structures with concrete or masonry wall systems	0.0075L	0.005L	0.002L
Other single-story structures	0.015L	0.010L	0.002L
Multistory structures with concrete or masonry wall systems	0.005L	0.003L	0.002L
Other multistory structures	0.010L	0.006L	0.002L

2.5.3 Bearing Failure/Surface Manifestations

We performed an analysis of the potential for bearing failures/structural damage due to liquefaction (surface manifestations) based on the work of Ishihara (1995) and as described in Martin and Lew (1999). This method is based on empirical data and considers the thickness of non-liquefiable soil below the ground surface and foundations, compared to the thickness of underlying liquefiable soils. Our analysis considers that proposed grade will be raised approximately 2 feet relative to the existing surface at the proposed restroom building location based on the grading plan. Based on our analysis and considering that the liquefiable layer is encountered at a depth of 11 feet below proposed grade and the upper soils are overexcavated and recompacted per our recommendations, the potential for structural damage due to liquefaction is low.

2.5.4 Seismically Induced Landslides

The site and its surroundings are relatively flat and do not have any significant slopes. The State of California has not evaluated the site for seismic landslide hazards. Additionally, the County of San Bernardino has mapped the site to be outside of a zone of Generalized Landslide Susceptibility. Given these considerations, the potential for seismically induced landslides to affect the site is not considered significant.

2.5.5 Surface Fault Rupture

The proposed development is not located within an Earthquake Fault Zone, as designated by the State of California or County of San Bernardino. Nor did available published geologic mapping identify the trace of any faults within or trending towards the site. Given the above, the surface fault rupture potential on the site is considered very low.

2.5.6 Flooding and Dam Breach Inundation

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site **is** located within a 500-year flood hazard zone.

Earthquake-induced flooding can be caused by the failure of dams or other water-retaining structures due to earthquakes. The site has not been mapped within a dam breach inundation zone (CDWR, 2023b).

2.5.7 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland site location, seiche and tsunami risks are considered negligible.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed restroom/shower building 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, presence of liquefiable soils, and potentially compressible soils. In order to mitigate potential differential settlements and liquefaction related lateral spread to tolerances that meet the project requirements, we recommend use of a geogrid-reinforced granular mat to provide support to shallow foundations.

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, San Bernardino County, 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 <u>Site Preparation</u>

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 has been cleared, the soils should be carefully observed for the removal of any unsuitable fill materials (if encountered) by a representative of the geotechnical engineer.

3.1.2 <u>Overexcavation, Geogrid and Recompaction</u>

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

For the proposed building pad area, existing soils should be removed to a minimum depth of 5 feet below current grades or 2 feet below bottom of proposed foundations, whichever is deeper. Deeper overexcavation may be recommended depending on exposed conditions during grading. Existing groundwater conditions should be considered during earthwork operations. Removal bottoms should extend horizontally a minimum of 5 feet beyond the outside edges of footings (including columns connected to buildings), or a distance equal to the depth of overexcavation below the footings, whichever is greater. In-place alluvial soils should be deemed suitable for new fill placement if possessing a minimum in situ relative compaction of 90 percent (ASTM Test Method D1557). Suitability of all removal bottoms should be reviewed and evaluated by an engineering geologist or a representative of the geotechnical engineer.

Geogrid: We recommend that the proposed restroom building structure be underlain by two layers of a Tensar TriAx TX160 triaxial geogrid. The first layer of geogrid should be placed on the recompacted removal bottom approximately 5 feet below existing grade. The first layer of geogrid should be laid at bottom of the overexcavation, extending a minimum of 5 feet beyond the proposed building foundation line and extending up the sides of the excavation with enough geogrid to allow for a minimum 10-foot fold-over return one foot above the second layer of geogrid. A 1-foot-thick layer of aggregate base should be placed over the first layer of geogrid; the aggregate base should be compacted to a minimum of 95 percent relative compaction per ASTM D1557. The second layer of geogrid and another 1-foot-thick layer of aggregate base should be placed over the initial base layer and extended to the edge of the excavation fill. The 5-foot return should then be placed over the second layer of compacted base. The remaining overexcavation backfill using onsite soils should then continue to design grade as recommended below. Additional geogrid construction considerations are presented in Section 3.8 of this report.

Areas outside of the proposed structures planned for new asphalt or concrete pavement (such as parking areas), flatwork (such as sidewalks), site walls and low retaining walls (3-foot retained height; taller walls should be overexcavated per the recommendations for buildings), areas to receive fill, and other improvements, should be overexcavated to a minimum depth of 18 inches below existing grade or 12 inches below proposed subgrade (including the footing subgrade for walls), 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.

3.1.3 Fill Placement and Compaction

Onsite soil may be used for compacted structural fill provided it is free of debris and oversized material (greater than 8 inches in largest dimension). Additionally, 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. 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.

3.1.4 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 13% +/- 5
Subsidence (overexcavation bottom processing)	Approximately 0.2 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 Oversized Material

Although materials larger than 4 inches in dimension were not recorded during logging of our small-diameter borings, rocks (larger than 12 inches in their largest dimension) may be encountered during grading, requiring special handling of these rocks or disposal offsite.

During fill placement, rocks larger than 12 inches in their largest dimension should be removed from within 3 feet of finish grade. If encountered during grading, no rocks larger than 24 inches should be placed within 10 feet of finish grade. All rocks larger than 24 inches in greatest dimension should be placed in windrows, surrounded with sandy soils and placed with copious amounts of water or disposed of properly. The rock windrows should be placed such that individual rocks are not nested and sandy soil can be worked completely around the rocks.

3.2 Shallow Foundation Recommendations

The proposed restroom building can be supported on shallow foundations. Maximum column loading and wall loading is not available at the time of this report. We have anticipated that the proposed restroom building will be lightly loaded. Structural loading information should be provided to us when available for review.

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

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment of 18 inches and maximum embedment of 24 inches, with a minimum width of 24 and 15 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 1,800 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. 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. This 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 0.35. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 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 0.1 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.

Considering the design PGA of 0.55g, and the minimum overexcavation recommendations are followed, potential differential seismic settlement is estimated to be up to a maximum of 2.5 inches, with an estimated potential differential settlement of 1.2 inches over 20 feet (angular distortion of 0.005L).

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a “low” expansion potential and considering the potential for seismically induced settlement. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture vapor retarder, steel or concrete.

- Moisture Retarder: A minimum 10-mil thick polyethylene moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced, 4-inch-thick slabs) should be No. 3 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Crack control joints should be provided at a maximum spacing of 15 feet on center.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

3.4 Retaining Wall Recommendations

We are not aware of retaining walls planned for the proposed improvements. The following recommendations are applicable for retaining walls shorter than 5 feet. 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)	240 (Maximum of 3,000 psf)

The above values do not contain an appreciable factor of safety unless note, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

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 0.35 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.

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 18 inches below the lowest adjacent grade. An allowable bearing capacity of 1,800 pcf may be used for retaining wall footing design, based on the minimum footing width and depth.

3.5 Sulfate Attack and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. The concrete should be designed in accordance with Table 19.3.2.1 of the American Concrete Institute ACI 318-14 provisions (ACI, 2014).

The onsite soil is considered to be severely corrosive to ferrous metals. It is recommended that any buried pipe be made of non-ferrous material, or that any ferrous pipe be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if metallic utilities are planned.

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 26H, 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-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, organic and oversized material. 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 will allow water to sufficiently permeate. Gravel or rock cannot be used for trench backfill without written approval by Leighton. If gravel or open-graded rock is approved and used as bedding or shading, it should be wrapped in Mirafi 140N filter fabric, or equivalent, to prevent surrounding soil from washing into the pore spaces in the gap graded rock. Shading should extend at least 12 inches above the top of the pipe. The bedding/shading materials should be densified in-place by mechanical means, or in accordance with Greenbook specifications.

Subsequent to pipe bedding and shading, backfill soils should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction (ASTM D1557). The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook). The upper 6 inches in pavement areas should be compacted to 95 percent compaction.

3.8 Geogrid Installation

All field handling and installation procedures should be performed in accordance with manufacturer's guidelines with a particular focus to ensure proper overlap between adjacent sheets, if specified by the manufacturer. Geogrid installation is also recommended to comply with the following:

- Geogrid rolls generally come in roll widths of 13 and 16 feet, either may be used.
- Geogrid reinforcement may be secured in-place with staples, pins, sand bags, or backfill as required by fill properties, fill placement procedures, or weather conditions, or as directed by the geotechnical engineer.

<u>Backfill Placement over Geogrids</u>

The placement of fill soils to finish grade should include certain procedures and precautions to protect the geogrid reinforcement and achieve proper recompaction. Fill placement and compaction is recommend to comply with the following:

- Backfill material (aggregate base) should be placed in thin lifts (4- to 6-inch thick) and compacted to a minimum of 95 percent per ASTM D1557. Actual lift thickness should be consistent with the equipment used for compaction.
- Backfill should be placed, spread and compacted in such a manner that minimizes the development of wrinkles/bends in and/or movement of the geogrid reinforcement.
- Care should be taken by the grading contractor that the fill soils and the grading equipment does not damage the integrity of the geogrid reinforcement during the construction process.
- Tracked construction equipment should not be operated directly upon the geogrid reinforcement. A minimum thickness of six (6) inches is required prior to operation of tracked vehicles over the geotextile reinforcement fabric. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the geogrid reinforcement.
- Rubber-tired equipment may pass over geogrid reinforcement at slow speeds, less than 5 mph. Sudden braking and sharp turning should be avoided.

If future excavations (such as utility trenching) will penetrate through the installed geogrid layer, then a cut geogrid section should be placed at the bottom of the utility trench extending the width of the existing geogrid. If both layers of geogrid are trenched through, a second layer should be placed close to the elevation of the adjacent top geogrid layer. Additional recommendations and considerations are provided within the Tensar TriAx Geogrid Installation Guide.

3.9 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.

3.10 Pavement Design Parameters

Flexible Pavements: Based on the design procedures outlined in the 2017 Caltrans Highway Design Manual, and using a maximum R-value of 20 based on laboratory testing, flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer.

ASPHALT PAVEMENT SECTION THICKNESS		
Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
5 or less (auto access)	3.0	7.5
7 (light truck access)	4.0	12.0

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 parking areas, we recommend a minimum of 5-inch-thick concrete, placed on a minimum 4 inches of aggregate base compacted to 95 percent relative compaction, over a minimum 6 inches of subgrade soils compacted to a minimum of 95 percent relative compaction.

The PCC pavement sections should be provided with crack-control joints spaced no more than 12 feet for 5-inch-thick concrete. 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: Irrigation adjacent to pavements without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure.

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 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

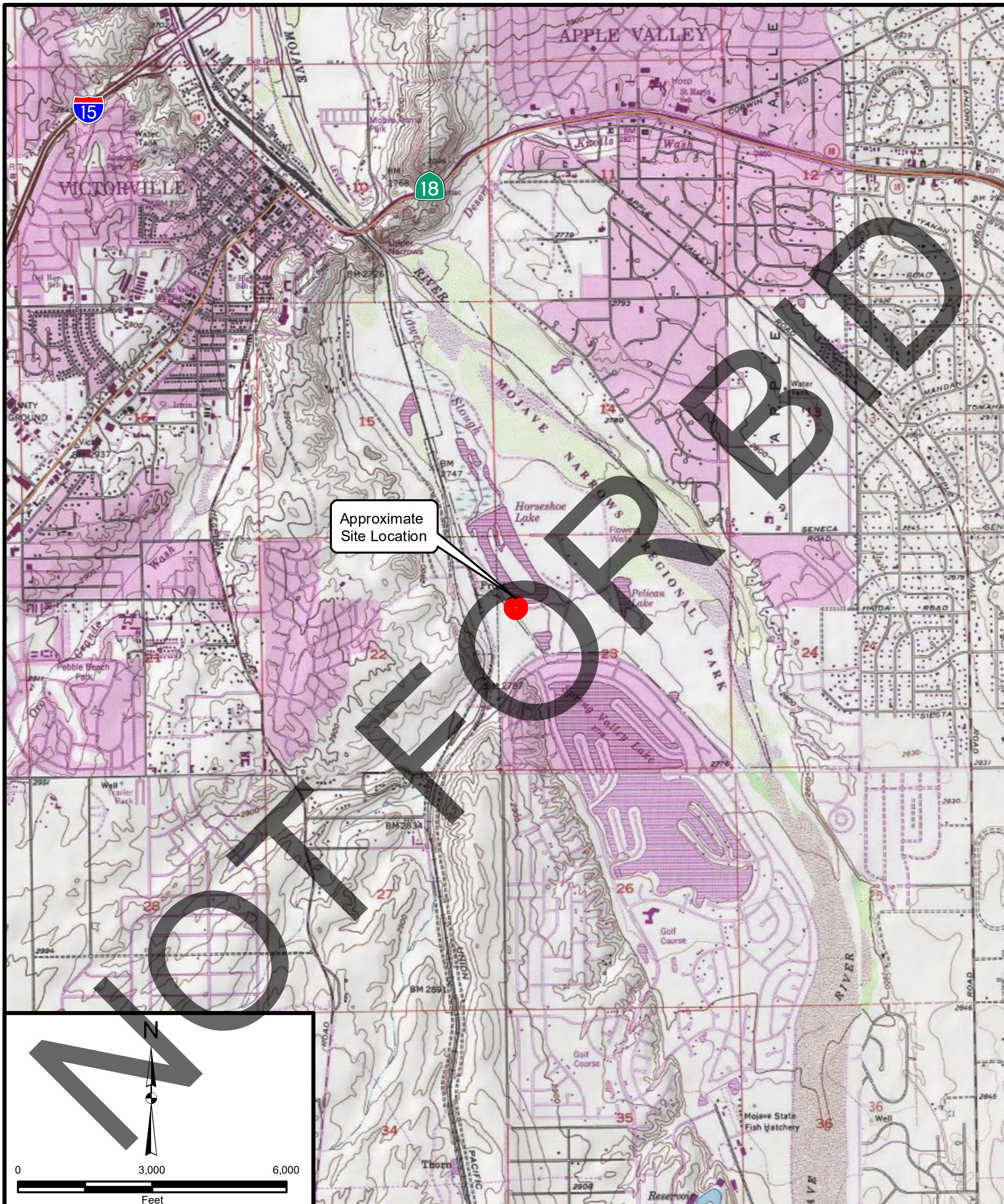
3.11 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from this limited subsurface explorations 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 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.



Project: 12099.006	Eng/Geol: JDH/SGO
Scale: 1 " = 3,000 '	Date: May 2023

Reference: Copyright:© 2013 National Geographic Society, i-cubed

SITE LOCATION MAP

Proposed Accessible Campsite Restroom, Mojave Narrows Regional Park, 18000 Yates Road, Unincorporated San Bernardino County, California

FIGURE 1

Leighton

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Project: 12099.006	Eng/Geol: JDH/SGO
Scale: 1 " = 30 '	Date: May 2023

Reference: © 2023 Microsoft Corporation © 2023 Maxar ©CNES (2023) Distribution Airbus DS © 2023 TomTom, Rexford Industrial Realty, Conceptual Sie Plan

GEOTECHNICAL EXPLORATION MAP

Proposed Accessible Campsite Restroom,
Mojave Narrows Regional Park, 18000 Yates Road,
Unincoporated San Bernardino County, California

FIGURE 2

Leighton

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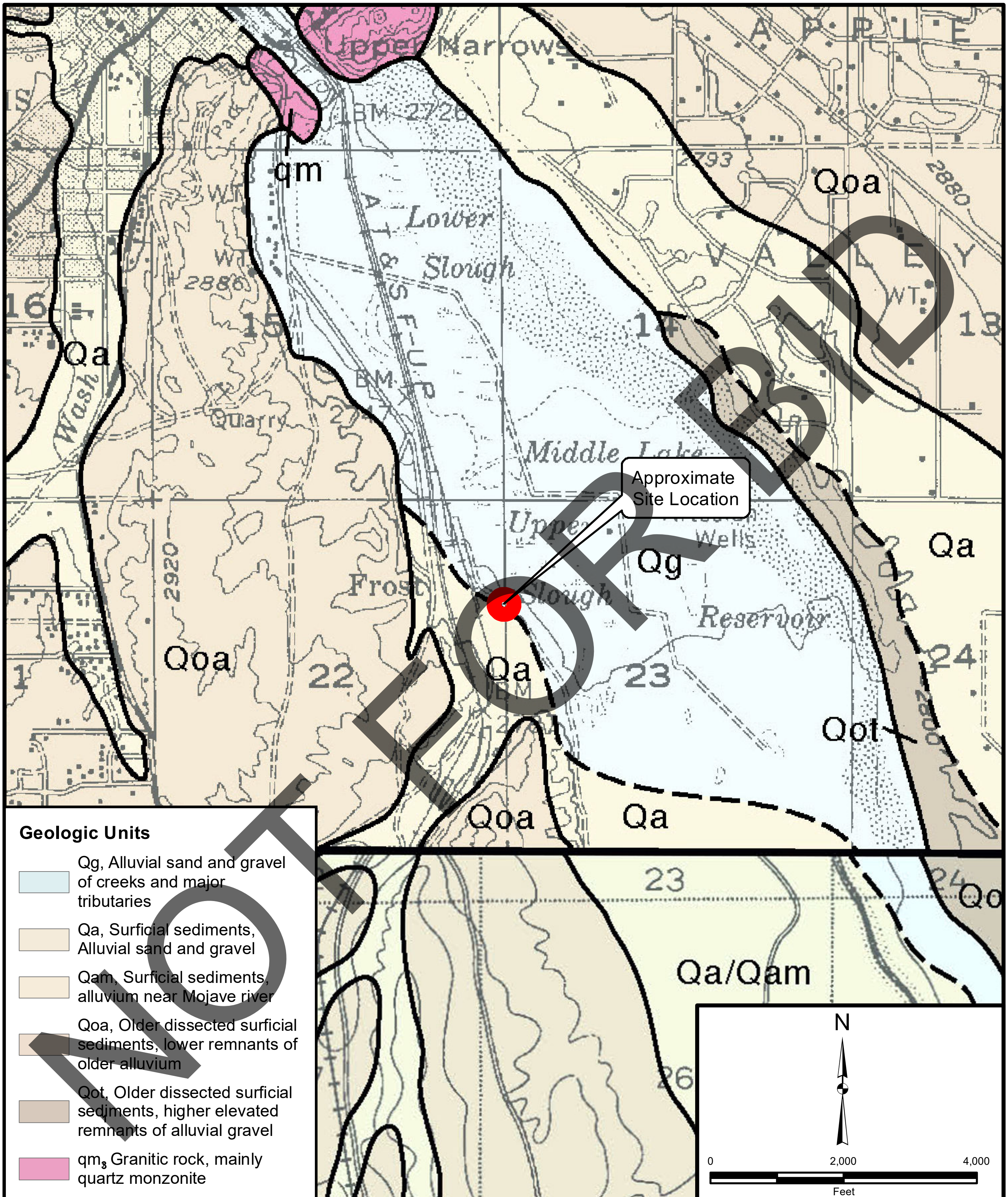

Project: 12099.006	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: May 2023

Reference: Geologic map of the Hisperia, Shadow Creek, and Victorville Quadrangles, Thomas W. Dibblee Jr

REGIONAL GEOLOGY MAP

Proposed Accessible Campsite Restroom,
Mojave Narrows Regional Park, 18000 Yates Road,
Unincorporated San Bernardino County, California

FIGURE 3

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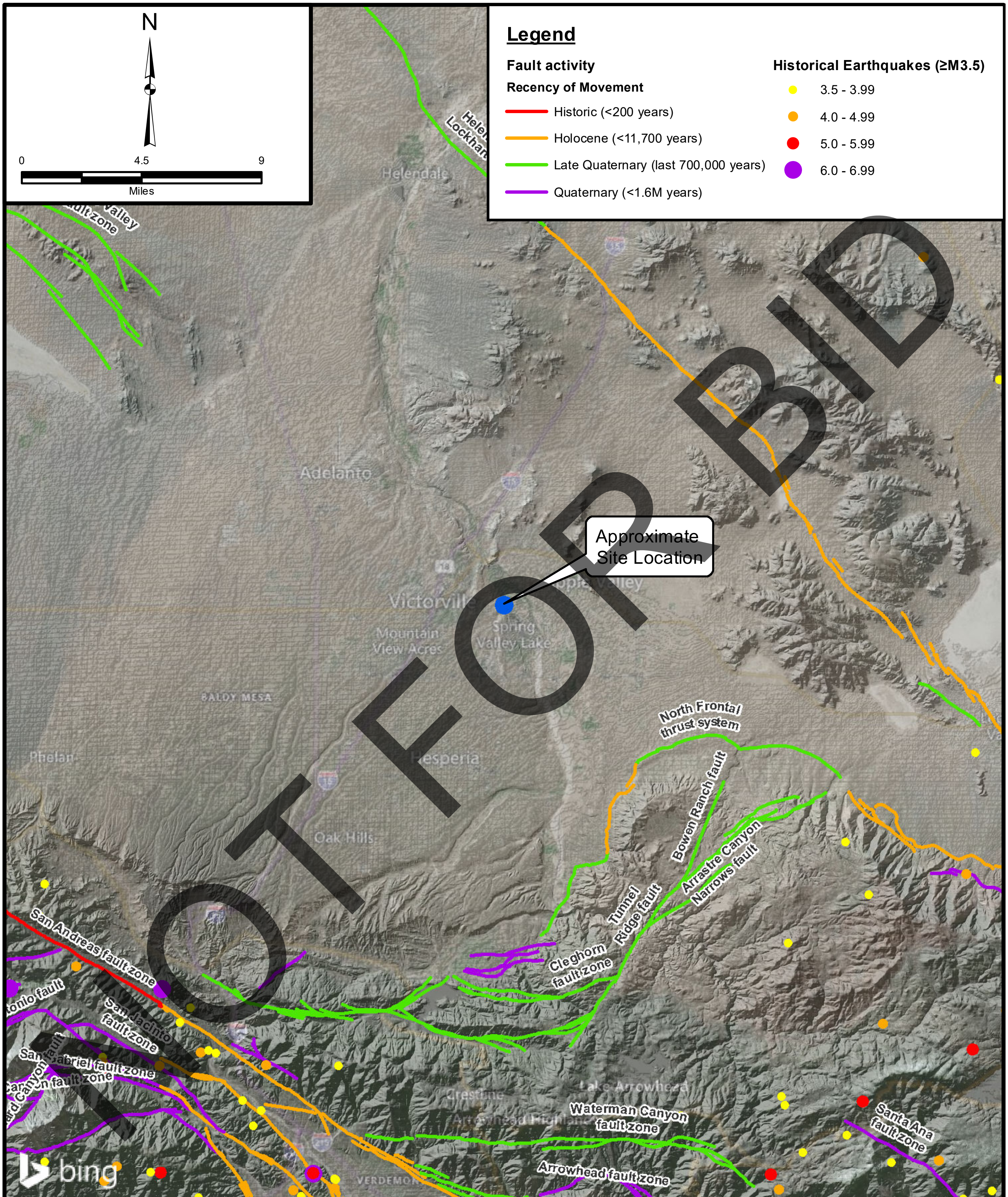

Project: 12099.006	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: May 2023

Basemap Reference: © 2023 Microsoft Corporation
Earthstar Geographics SIO © 2023 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Proposed Accessible Campsite Restroom, Mojave
Unincorporated San Bernardino County, California

FIGURE 4

Leighton

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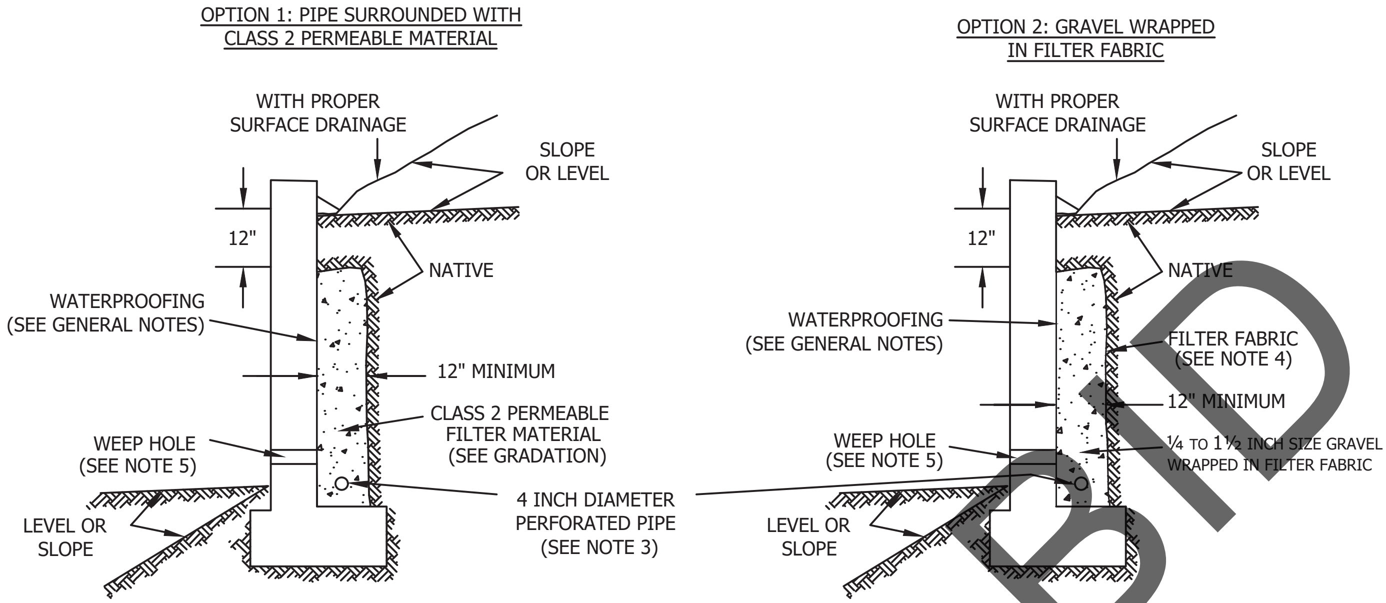
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



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

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Figure 5

APPENDIX A

REFERENCES

APPENDIX A

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APPENDIX B

GEOTECHNICAL EXPLORATION LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	12099.006	**Date Drilled**	4-20-23
Project	Mojave Narrows Regional Park Restroom GE	**Logged By**	AA
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer - 30"" Drop	**Ground Elevation**	2761'
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 *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.*	Type of Tests
	0			B-1				SM	**Quaternary Alluvium, undifferentiated**	SA, MD, DS, EI
2760									@Surface: GRASS over SILTY SAND (SM), brown, dry, fine to medium sand, 30% fines (field estimate)	
				R-1	2 5 7	86	20	ML	@2.5': SILT (ML), firm, brown, slightly moist, rapid dilatency, 95% low plasticity fines, (field estimate)	
2755	5			R-2	3 6 7	98	23	SC	@5': CLAYEY SAND (SC), loose, brown, moist, fine sand, 47% fines (lab)	-200, AL
				R-3	4 6 7	100	23	SP	@7.5': POORLY GRADED SAND (SP), olive, WET, medium to coarse sand, 4% fines (field estimate)	
								SM	@8.75': SILTY SAND (SM), loose, dark gray, WET, olive, very fine sand, 40% fines (field estimate) @9'2": Groundwater measured inside auger	
2750	10			R-4	4 7 6	109	11	SP-SM	@10': POORLY GRADED SAND with SILT (SP-SM), loose, dark gray, WET, olive, very fine sand, 5% fines (lab)	-200
2745	15			S-1	2 4 4			SW	@15': WELL GRADED SAND (SW), loose, olive brown, WET, coarse sand, 1% fines (field estimate)	-200, CO
2740	20			R-5	10 25 26	124	14	SP-SM	@21': POORLY GRADED SAND WITH SILT (SP-SM), dense, olive, WET, coarse sand, 10% fines (field estimate) -Small pocket of SILTY SAND (30% fines) in center of sample	
2735	25			S-2	12 20 20			SP-SM	@25': POORLY GRADED SAND WITH SILT (SP-SM), dense, olive, WET, coarse sand, trace of gravel, 10% fines (field estimate) -Heavy auger chatter from 25 to 30 feet	
	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

Leighton

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-1

Project No.	12099.006	**Date Drilled**	4-20-23
Project	Mojave Narrows Regional Park Restroom GE	**Logged By**	AA
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer - 30"" Drop	**Ground Elevation**	2761'
Location	See Figure 2 - Exploration Location Map	**Sampled By**	AA

SOIL DESCRIPTION

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
2730	30			S-3	11 24 34			SP	@30': POORLY GRADED SAND with GRAVEL (SP), very dense, olive, WET, coarse sand, 20% gravel, 4% fines (field estimate) -Heavy auger chatter from 30 to 33 feet	
2725	35			S-4	3 5 11			CL	@35': SANDY LEAN CLAY (CL), stiff, brown, WET, 60% medium plasticity fines (field estimate)	
2720	40			S-5	4 6 11			CL	@40': SANDY LEAN CLAY (CL), very stiff, brown, sightly moist, high dry strength, 62% medium plasticity fines (lab)	-200, AL
2715	45			S-6	8 11 14			CL	@45': SANDY LEAN CLAY (CL), very stiff, brown, moist, 60% medium plasticity fines (field estimate)	
2710	50			S-7	9 16 21			CL	@50': SANDY LEAN CLAY (CL), hard, brown, moist, 60% medium plasticity fines (field estimate) @51': SANDY LEAN CLAY (CL), hard, brown, moist, medium to coarse sand, 70% medium plasticity fines (field estimate)	
2705	55									
	60									

TOTAL DEPTH = 51.5 FEET
GROUNDWATER ENCOUNTERED AT APPROXIMATELY 9 FEET DURING DRILLING
HOLE COLLAPSED TO 9.5 FEET BGS AFTER PULLING AUGER OUT
GROUNDWATER MEASURED AT 6.5 FEET AFTER PULLING AUGERS OUT

BACKFILLED TO SURFACE WITH SOIL CUTTINGS

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.	12099.006	**Date Drilled**	4-20-23
Project	Mojave Narrows Regional Park Restroom GE	**Logged By**	AA
Drilling Co.	Martini Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - Autohammer - 30"" Drop	**Ground Elevation**	2760'
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 *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.*	Type of Tests
2760	0							SM	**Quaternary Alluvium, undifferentiated** @Surface: GRASS over SILTY SAND (SM), brown, dry, fine to medium sand, 30% fines (field estimate)	
				R-1	3 7 11			SC	@2.5': CLAYEY SAND (SC), medium dense, dark brown, slightly moist, 45% fines (field estimate)	
2755	5			R-2	4 7 8			SC	@5': CLAYEY SAND (SC), loose, orange brown, slightly moist, medium to coarse sand, 45% fines (field estimate)	
				R-3	4 6 7	92	29	SP-SM	@7.5': POORLY GRADED SAND WITH SILT (SP-SM), loose, dark gray, moist, olive, very fine sand, 5% fines (field estimate) @9': Groundwater measured inside auger	
2750	10			R-4	4 7 6	86	33	SP-SM	@10': POORLY GRADED SAND WITH SILT (SP-SM), loose, dark gray, moist, olive, very fine sand, 5% fines (field estimate)	
2745	15			S-1	1 2 4			SP	@15': POORLY GRADED SAND (SP), loose, gray, WET, coarse sand, 4% fines (field estimate)	
2740	20								**TOTAL DEPTH = 16.5 FEET** **GROUNDWATER ENCOUNTERED AT APPROXIMATELY 9 FEET DURING DRILLING** **HOLE COLLAPSED TO 5 FEET 7 INCHES BGS AFTER PULLING AUGER OUT** **GROUNDWATER MEASURED AT 9 FEET AFTER PULLING AUGERS OUT** **BACKFILLED TO SURFACE WITH SOIL CUTTINGS**	
2735	25									
2730	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

Leighton

NOT FOR BID

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

APPENDIX C

LABORATORY TEST RESULTS

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MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Mojave Narrows Regional Park Restroom	Tested By: J. Foltz	Date: 05/08/23
Project No.:	12099.006	Input By: M. Vinet	Date: 05/10/23
Boring No.:	LB-1	Depth (ft.): 0 - 5.0	
Sample No.:	B-1		
Soil Identification:	Sandy Silt s(ML), Dark Yellowish Brown.		

Preparation Method: [X] Moist [ ] Dry [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03340 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5416	5502	5513			
Weight of Mold (g)	3522	3522	3522			
Net Weight of Soil (g)	1894	1980	1991			
Wet Weight of Soil + Cont. (g)	1623.2	1458.2	1559.2			
Dry Weight of Soil + Cont. (g)	1490.0	1320.9	1391.0			
Weight of Container (g)	277.8	276.2	280.2			
Moisture Content (%)	11.0	13.1	15.1			
Wet Density (pcf)	125.0	130.7	131.4			
Dry Density (pcf)	112.6	115.5	114.1			

Maximum Dry Density (pcf) **115.5** **Optimum Moisture Content (%)** **13.5**

PROCEDURE USED

[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

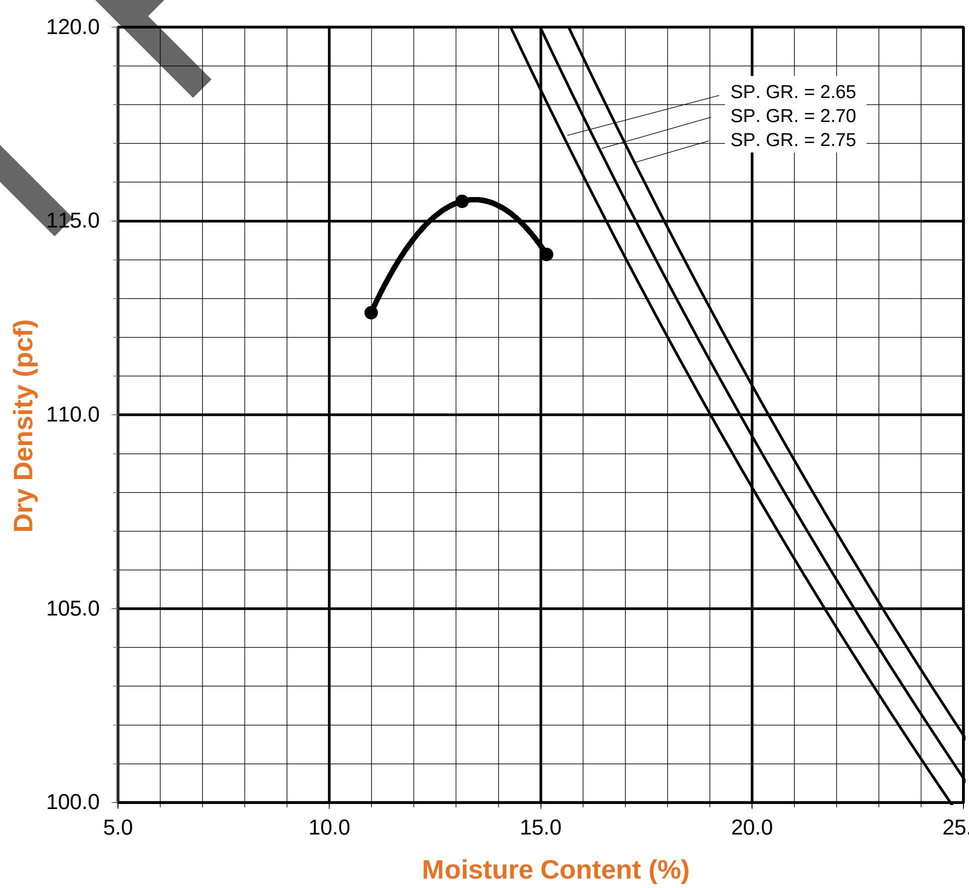
[ ] **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

[ ] **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:
0:42:58
GR:SA:FI

Atterberg Limits:

LL,PL,PI



Compaction; LB-1, B-1 (04-20-23)

Boring No.	LB-1	LB-1	LB-1	LB-1				
Sample No.	R-2	R-4	S-1	S-5				
Depth (ft.)	5.0	10.0	15.0	40.0				
Sample Type	RING	RING	SPT	SPT				
Soil Classification	SC	SP-SM	SW	s(CL)				
Soak Time (min)	10	10	10	10				
Moisture Correction								
Wet Weight of Soil + Container (gm.)	711.1	715.2	759.8	559.2				
Dry Weight of Soil + Container (gm.)	631.1	609.4	673.7	507.8				
Weight of Container (gm)	280.6	281.6	278.9	278.1				
Moisture Content (%)	22.8	32.3	21.8	22.4				
Container No.:	MA	LA	BL	MAG				
Sample Dry Weight Determination								
Weight of Sample + Container (gm.)	631.1	609.4	673.7	507.8				
Weight of Container (gm.)	280.6	281.6	278.9	278.1				
Weight of Dry Sample (gm.)	350.5	327.8	394.8	229.7				
Container No.:	MA	LA	BL	MAG				
After Wash								
Dry Weight of Sample + Container (gm)	468.1	593.8	670.3	366.5				
Weight of Container (gm)	280.6	281.6	278.9	278.1				
Dry Weight of Sample (gm)	187.5	312.2	391.4	88.4				
% Passing No. 200 Sieve	**47**	**5**	**1**	**62**				
% Retained No. 200 Sieve	53	95	99	38				

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Mojave Narrows Regional Park Restroom
Project No.: 12099.006
Client Name: San Bernardino County Project and Facilities Management
Tested By: M. Vinet Date: 05/04/23

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name:	Mojave Narrows Regional Park Restroom	Tested By:	MRV	Date:	05/08/23
Project No.:	12099.006	Checked By:	MRV	Date:	08/10/23
Boring No.:	LB-1	Depth (feet):	0 - 5.0		
Sample No.:	B-1				
Soil Identification:	Sandy Silt s(ML), Dark Yellowish Brown.				

		Moisture Content of Total Air - Dry Soil	
Container No.:	LB	Wt. of Air-Dry Soil + Cont. (g)	828.4
Wt. of Air-Dried Soil + Cont.(g)	828.4	Wt. of Dry Soil + Cont. (g)	769.0
Wt. of Container (g)	278.1	Wt. of Container No.______ (g)	278.1
Dry Wt. of Soil (g)	490.9	Moisture Content (%)	12.1

After Wet Sieve	Container No.	LB
	Wt. of Dry Soil + Container (g)	514.5
	Wt. of Container (g)	278.1
	Dry Wt. of Soil Retained on # 200 Sieve (g)	236.4

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
3"	75.000		100.0
1"	25.000		100.0
3/4"	19.000		100.0
1/2"	12.500		100.0
3/8"	9.500		100.0
#4	4.750	0.0	100.0
#8	2.360	5.7	98.8
#16	1.180	22.6	95.4
#30	0.600	48.6	90.1
#50	0.300	88.3	82.0
#100	0.150	133.5	72.8
#200	0.075	205.8	58.1
PAN			

GRAVEL: **0 %**

SAND: **42 %**

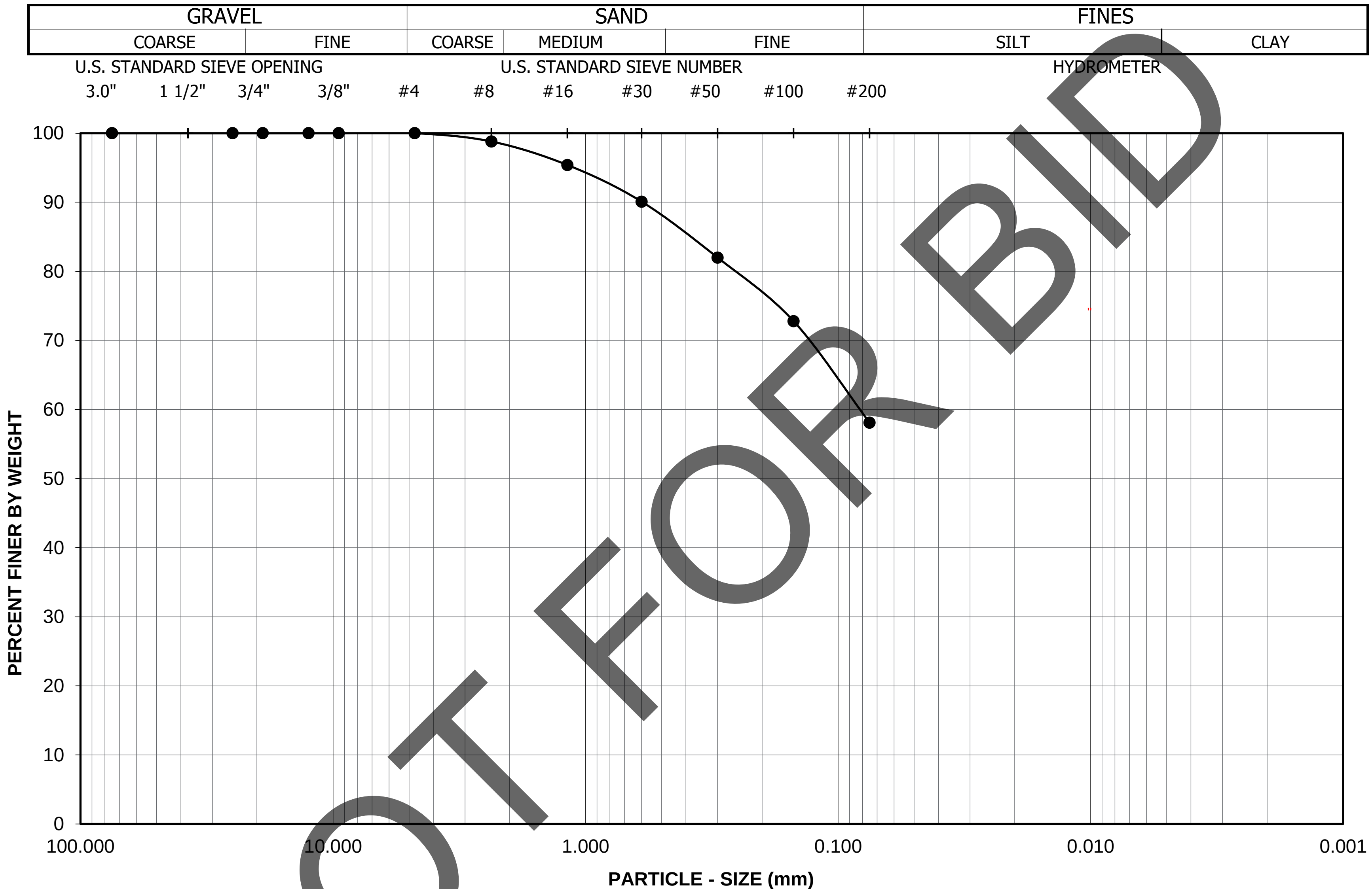
FINES: **58 %**

GROUP SYMBOL: **s(ML)**

Cu = D60/D10 = N/A

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

Remarks:



Project Name: Mojave Narrows Regional Park Restroom

Project No.: 12099.006

Boring No.: LB-1

Sample No.: B-1

Depth (feet): 0 - 5.0

Soil Type : s(ML)

Soil Identification: Sandy Silt s(ML), Dark Yellowish Brown.

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 42 : 58

Aug-23

Sieve; LB-1, B-1 (04-20-23)

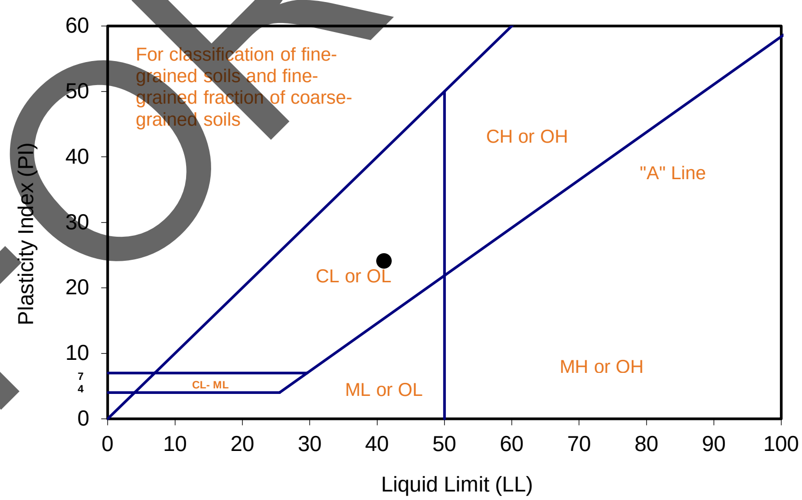
ATTERBERG LIMITS

ASTM D 4318

Project Name:	Mojave Narrows Regional Park Restroom	Tested By:	F. Mina	Date:	05/10/23
Project No. :	12099.006	Input By:	M. Vinet	Date:	05/11/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	R-2	Depth (ft.)	5.0		
Soil Identification:	Clayey Sand (SC), Dark Brown.				

TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			15	25	35	
Wet Wt. of Soil + Cont. (g)	22.48	22.99	22.32	21.50	23.02	
Dry Wt. of Soil + Cont. (g)	21.21	21.65	19.75	19.23	20.42	
Wt. of Container (g)	13.66	13.76	13.78	13.65	13.77	
Moisture Content (%) [Wn]	16.82	16.98	43.05	40.68	39.10	

Liquid Limit	**41**
Plastic Limit	**17**
Plasticity Index	**24**
Classification	**CL**



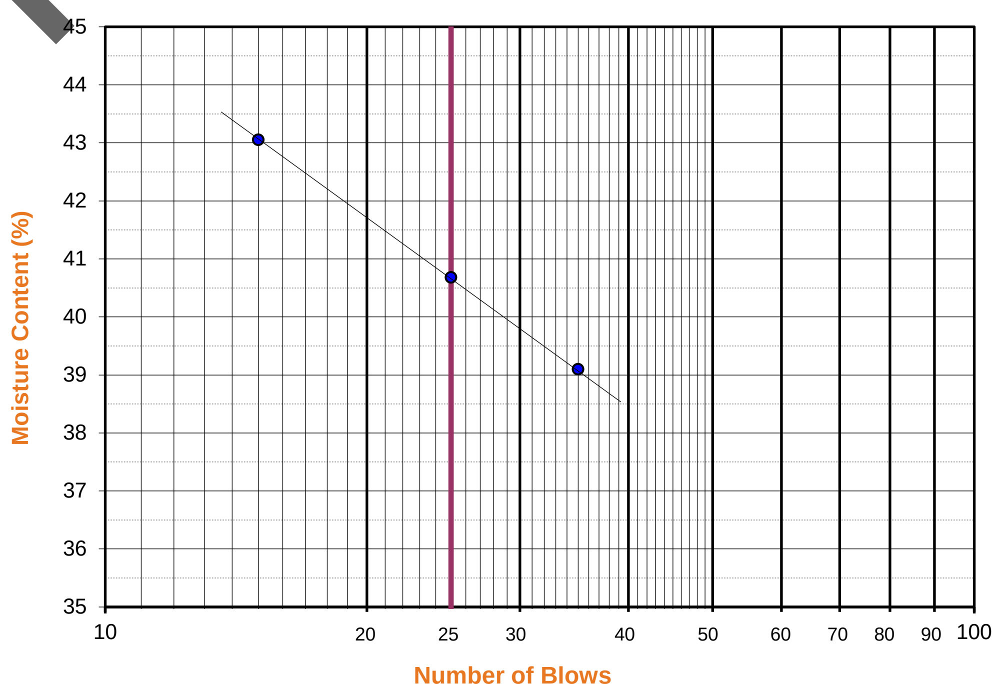
PI at "A" - Line = 0.73(LL-20) | 15.33

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$

PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



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ATTERBERG LIMITS

ASTM D 4318

Project Name:	Mojave Narrows Regional Park Restroom	Tested By:	F. Mina	Date:	05/10/23
Project No. :	12099.006	Input By:	M. Vinet	Date:	05/11/23
Boring No.:	LB-1	Checked By:	M. Vinet		
Sample No.:	S-5	Depth (ft.)	40.0		
Soil Identification:	Sandy Lean Clay s(CL), Dark Brown.				

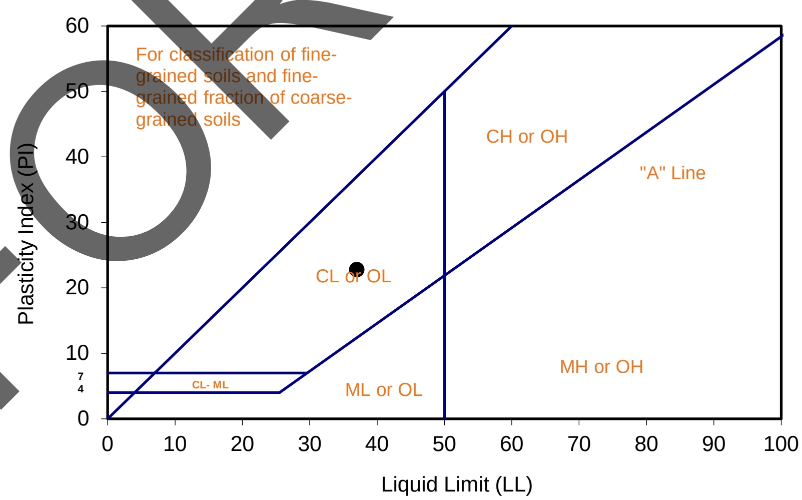
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			17	25	35	
Wet Wt. of Soil + Cont. (g)	18.73	20.23	23.63	20.12	21.67	
Dry Wt. of Soil + Cont. (g)	18.11	19.41	20.91	18.37	19.57	
Wt. of Container (g)	13.72	13.68	13.82	13.61	13.63	
Moisture Content (%) [Wn]	14.12	14.31	38.36	36.76	35.35	

Liquid Limit	**37**
Plastic Limit	**14**
Plasticity Index	**23**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) 12.41

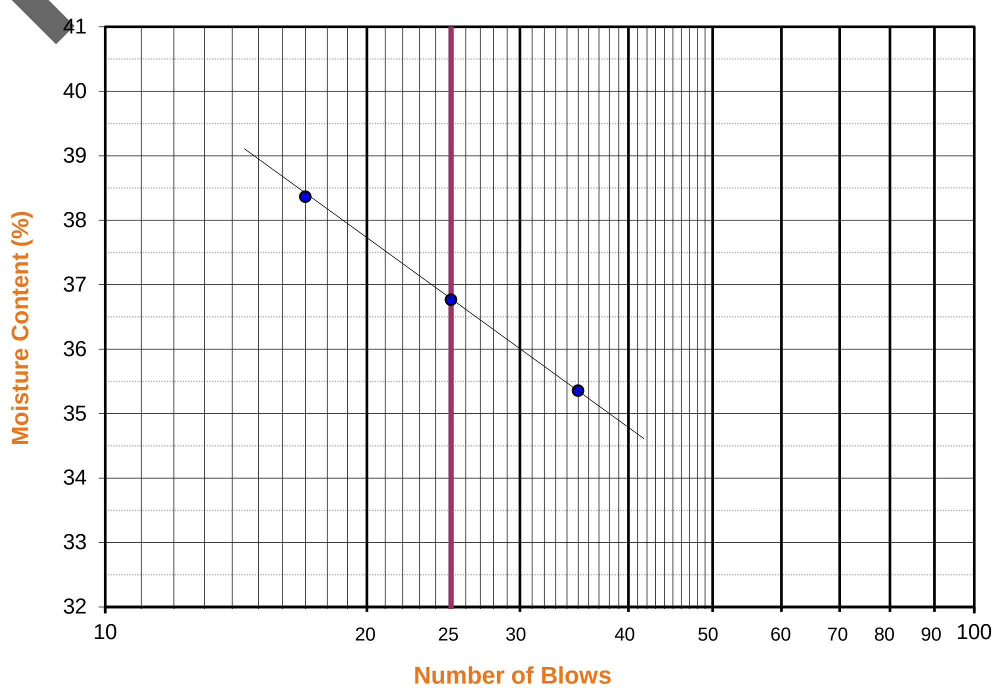
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Mojave Narrows Regional Park Restroom	Tested By:	M. Vinet	Date:	5/8/23
Project No. :	12099.006	Checked By:	M. Vinet	Date:	5/10/23
Boring No.:	LB-1	Depth:	0 - 5.0		
Sample No. :	B-1	Location:	N/A		
Sample Description:	Sandy Silt s(ML), Dark Yellowish Brown.				

Dry Wt. of Soil + Cont. (gm.)	490.9
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	490.9
Weight Soil Retained on #4 Sieve	0.0
Percent Passing # 4	100.0

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0276
Wt. Comp. Soil + Mold (gm.)	585.2	617.7
Wt. of Mold (gm.)	200.4	200.4
Specific Gravity (Assumed)	2.70	2.70
Container No.	8	8
Wet Wt. of Soil + Cont. (gm.)	577.7	617.7
Dry Wt. of Soil + Cont. (gm.)	545.3	343.3
Wt. of Container (gm.)	277.7	200.4
Moisture Content (%)	12.1	21.6
Wet Density (pcf)	116.1	122.5
Dry Density (pcf)	103.5	100.8
Void Ratio	0.628	0.673
Total Porosity	0.386	0.402
Pore Volume (cc)	79.9	85.6
Degree of Saturation (%) [S meas]	**52.0**	**86.5**

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.)
5/8/23	13:00	1.0	0	0.5000
5/8/23	13:10	1.0	10	0.5000
	Add Distilled Water to the Specimen			
5/9/23	8:00	1.0	1130	0.5276
5/9/23	9:00	1.0	1190	0.5276

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	27.6
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Heigh	**28**

Rev. 03-08

One-Dimensional Swell or Settlement Potential of Cohesive Soils
(ASTM D 4546) -- Method 'B'

Project Name: Mojave Narrows Regional Park Restroom | Tested By: M. Vinet | Date: 5/8/23
Project No.: 12099.006 | Checked By: M. Vinet | Date: 5/10/23
Boring No.: LB-1 | Sample Type: IN SITU
Sample No.: R-5 | Depth (ft.) 21.0
Sample Description: Well-Graded Sand with Silt (SW-SM), Yellowish Brown.
Source and Type of Water Used for Inundation: Arrowhead (Distilled)
** Note: Loading After Wetting (Inundation) not Performed Using this Test Method.

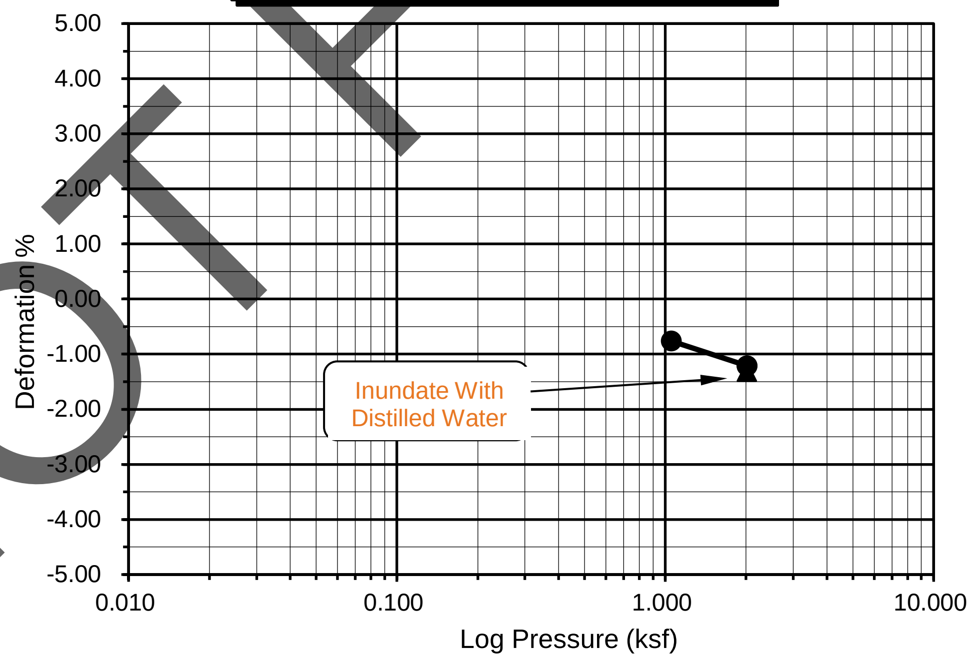
Initial Dry Density (pcf):	115.8
Initial Moisture (%):	16.8
Initial Height (in.):	1.0000
Initial Dial Reading (in):	0.0000
Inside Diameter of Ring (in):	2.416

Final Dry Density (pcf):	117.3
Final Moisture (%) :	17.0
Initial Void ratio:	0.4561
Specific Gravity (assumed):	2.70
Initial Degree of Saturation (%):	99.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
1.050	0.0076	0.9924	0.00	-0.76	0.4451	-0.76
2.013	0.0121	0.9879	0.00	-1.21	0.4385	-1.21
H2O	0.0132	0.9868	0.00	-1.32	0.4369	-1.32

Percent Swell / Settlement After Inundation = -0.11

Deformation % - Log Pressure Curve



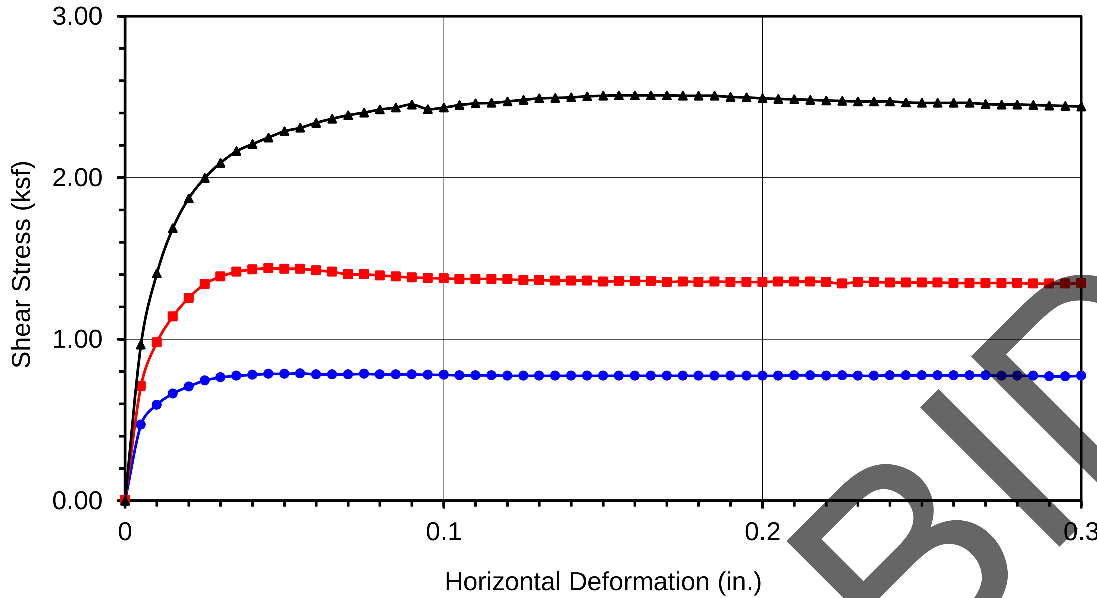
Rev. 01-10

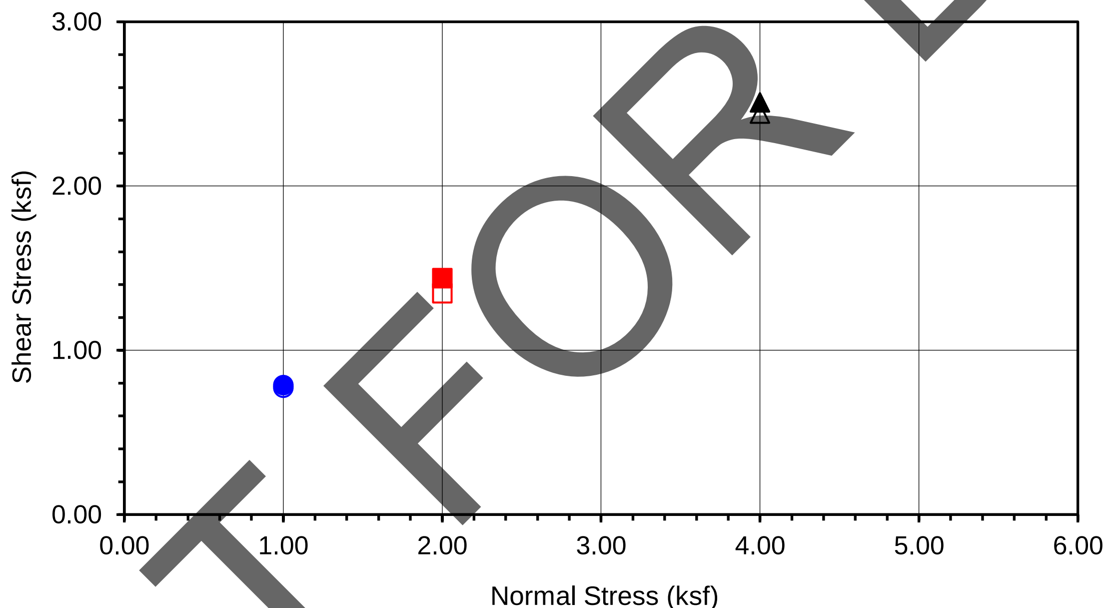
DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Mojave Narrows Regional Park Restroom	Tested By:	M. Vinet	Date:	05/08/23
Project No.:	12099.006	Checked By:	M. Vinet	Date:	05/10/23
Boring No.:	LB-1	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0 - 5.0		
Soil Identification:	Sandy Silt s(ML), Dark Yellowish Brown.				

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	183.01	185.56	186.27
Weight of Ring(gm):	41.97	42.90	43.65
Before Shearing			
Weight of Wet Sample+Cont.(gm):	288.79	288.79	288.79
Weight of Dry Sample+Cont.(gm):	259.75	259.75	259.75
Weight of Container(gm):	50.07	50.07	50.07
Vertical Rdg.(in): Initial	0.0000	0.2500	0.2500
Vertical Rdg.(in): Final	-0.0032	0.2583	0.2703
After Shearing			
Weight of Wet Sample+Cont.(gm):	202.57	201.51	201.98
Weight of Dry Sample+Cont.(gm):	175.68	176.34	177.07
Weight of Container(gm):	50.96	50.44	50.52
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0 - 5.0**
Sample Type: 90% Remold	
Soil Identification: Sandy Silt s(ML), Dark Yellowish Brown.	

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.788	■ 1.439	▲ 2.510
Shear Stress @ End of Test (ksf)	○ 0.773	□ 1.348	△ 2.441
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	13.85	13.85	13.85
Dry Density (pcf)	103.0	104.2	104.2
Saturation (%)	58.8	60.6	60.5
Soil Height Before Shearing (in.)	0.9968	0.9917	0.9797
Final Moisture Content (%)	21.6	20.0	19.7

Leighton

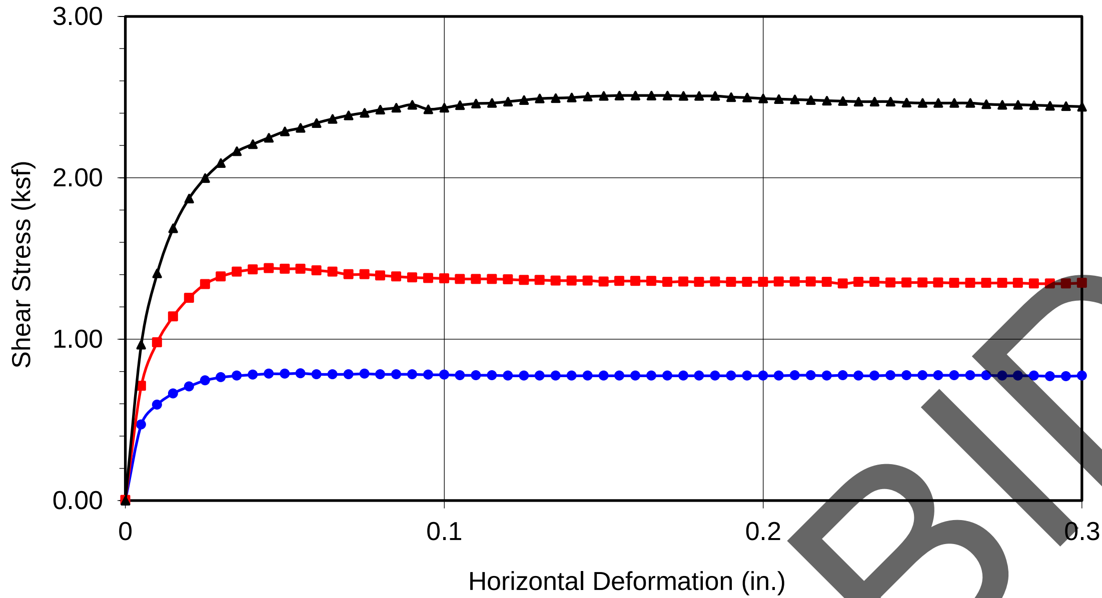
DIRECT SHEAR TEST RESULTS

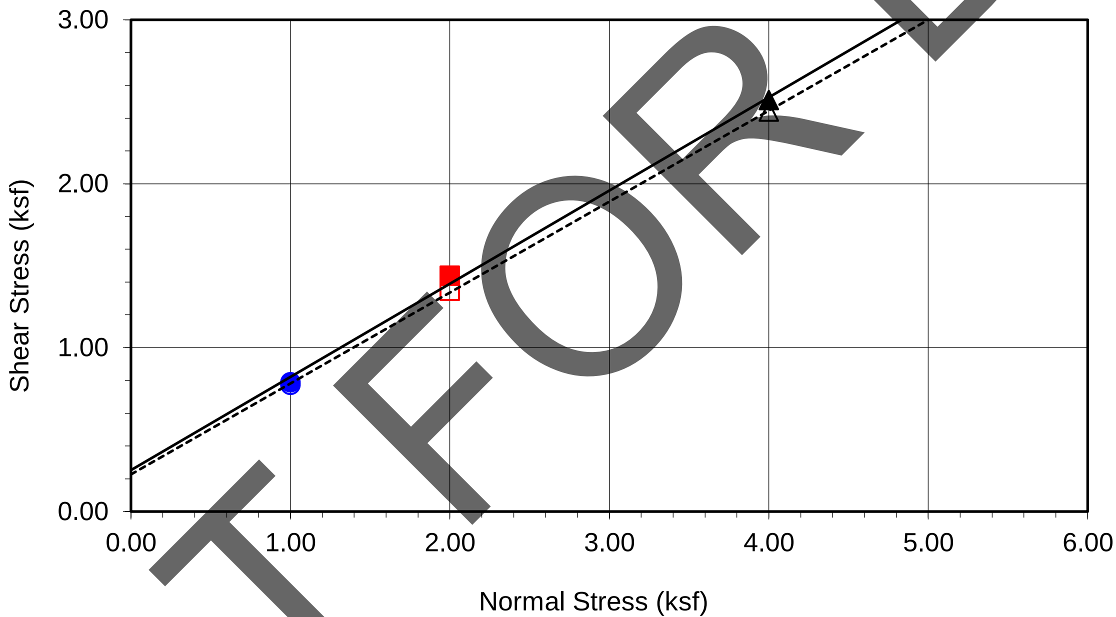
Consolidated Drained - ASTM D 3080

Project No.: 12099.006

Mojave Narrows Regional Park Restroom

05-23





Boring No.	**LB-1**
Sample No.	**B-1**
Depth (ft)	**0 - 5.0**
Sample Type:	90% Remold

Soil Identification:
Sandy Silt s(ML), Dark Yellowish Brown.

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	253	30
Ultimate	227	29

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.788	■ 1.439	▲ 2.510
Shear Stress @ End of Test (ksf)	○ 0.773	□ 1.348	△ 2.441
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	13.85	13.85	13.85
Dry Density (pcf)	103.0	104.2	104.2
Saturation (%)	58.8	60.6	60.5
Soil Height Before Shearing (in.)	0.9968	0.9917	0.9797
Final Moisture Content (%)	21.6	20.0	19.7

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 12099.006

Mojave Narrows Regional Park Restroom

05-23

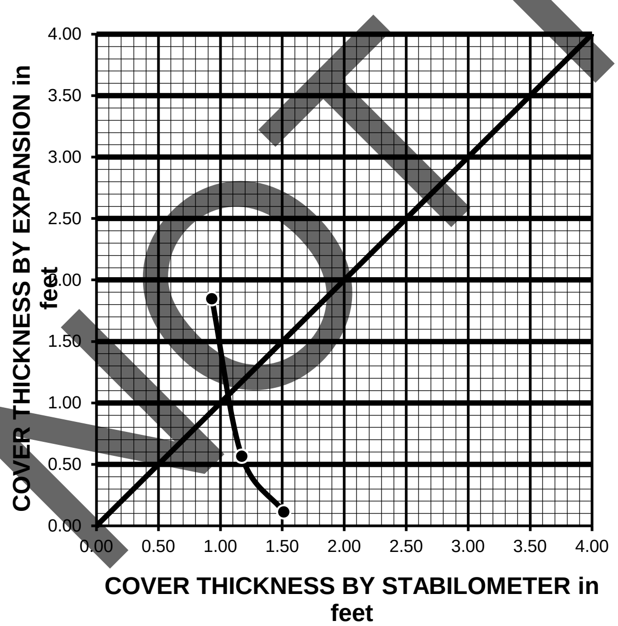
R-VALUE TEST RESULTS
ASTM D 2844

Project Name:	Mojave Narrows Regional Park Restroom	Date:	5/8/23
Project Number:	12099.006	Technician:	F. Mina
Boring Number:	LB-1	Depth (ft.):	0 - 5.0
Sample Number:	B-1		
Sample Description:	Sandy Silt s(ML), Dark Yellowish Brown.	Sample Location:	N/A

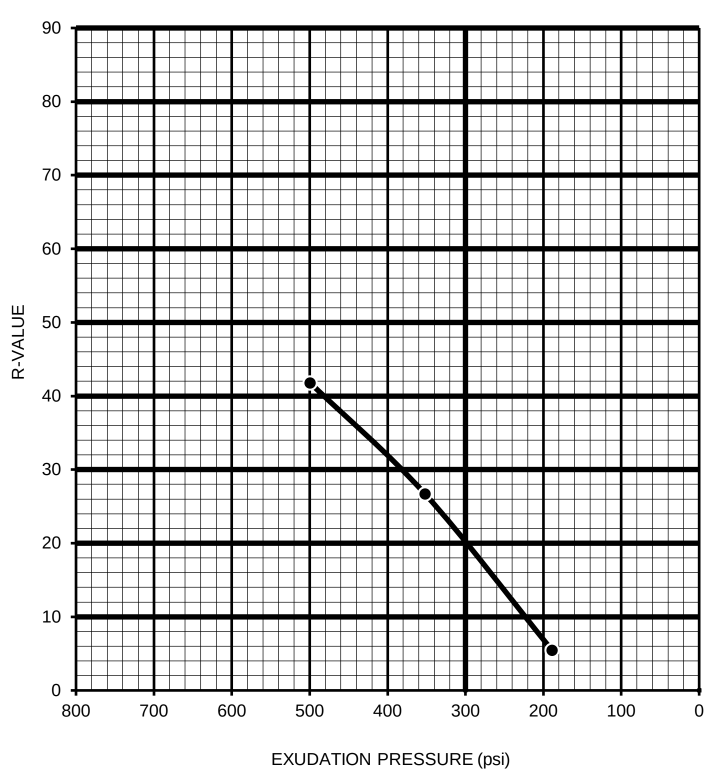
TEST SPECIMEN	A	B	C
MOISTURE AT COMPACTION %	14.3	15.5	16.6
HEIGHT OF SAMPLE, Inches	2.50	2.52	2.55
DRY DENSITY, pcf	104.9	104.4	104.3
COMPACTOR AIR PRESSURE, psi	150	125	90
EXUDATION PRESSURE, psi	499	352	189
EXPANSION, Inches x 10exp-4	49	15	3
STABILITY Ph 2,000 lbs (160 psi)	70	94	143
TURNS DISPLACEMENT	4.48	4.82	5.15
R-VALUE UNCORRECTED	42	27	5
R-VALUE CORRECTED	42	27	5

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.	0.93	1.17	1.51
EXPANSION PRESSURE THICKNESS, ft.	1.85	0.57	0.11

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	34
R-VALUE BY EXUDATION:	20
EQUILIBRIUM R-VALUE:	20

TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Mojave Narrows Regional Park Restroom	Tested By :	M. Vinet	Date:	05/09/23
Project No. :	12099.006	Data Input By:	M. Vinet	Date:	05/10/23

Boring No.	LB-1			
Sample No.	B-1			
Sample Depth (ft)	0 - 5.0			
Soil Identification:	Sandy Silt s(ML)			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	25.0532			
Wt. of Crucible (g)	25.0362			
Wt. of Residue (g) (A)	0.0170			
PPM of Sulfate (A) x 41150	699.55			
PPM of Sulfate, Dry Weight Basis	**700**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of $AgNO_3$ Soln. Used in Titration (C)	3.2			
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	**7.40**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 643

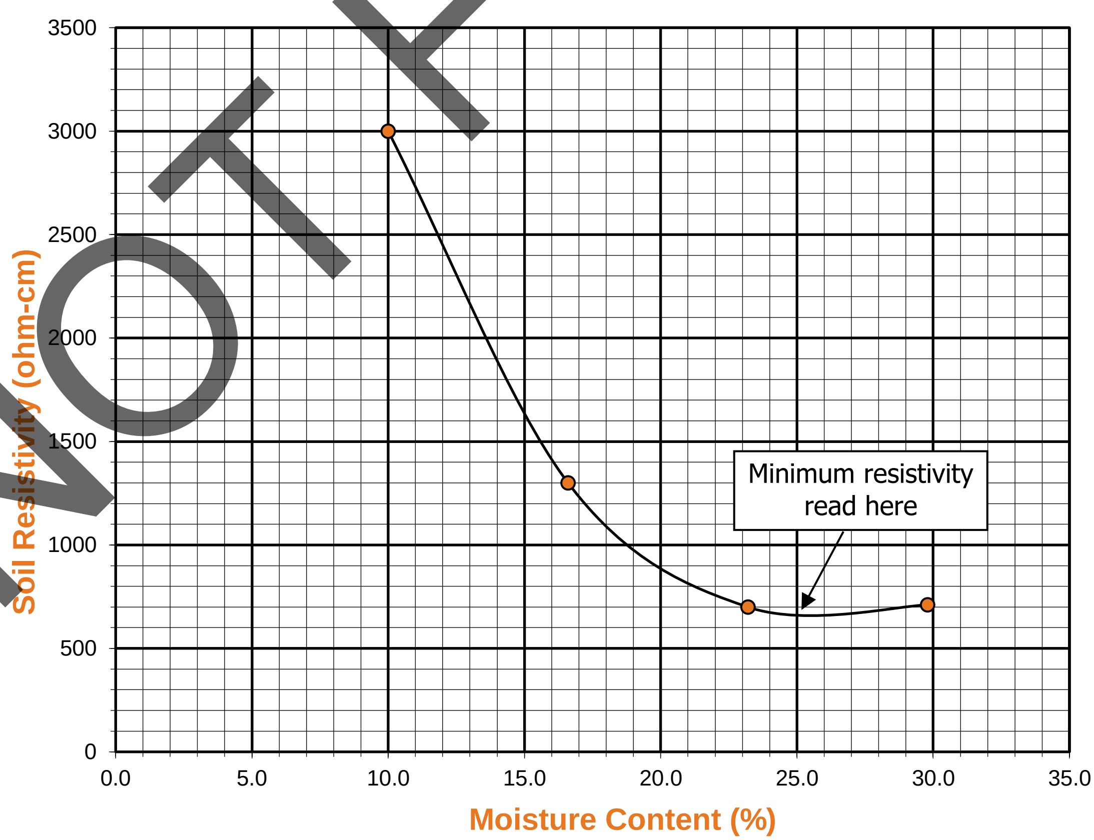
Project Name:	Mojave Narrows Regional Park Restroom	Tested By :	M. Vinet Date: 05/09/23
Project No. :	12099.006	Data Input By:	M. Vinet Date: 05/10/23
Boring No.:	LB-1	Depth (ft.) :	0 - 5.0
Sample No. :	B-1		
Soil Identification:*	Sandy Silt s(ML)		

*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	50	10.00	3000	3000
2	83	16.60	1300	1300
3	116	23.20	700	700
4	149	29.80	710	710
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
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	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
650	**25.0**	**700**	**300**	**7.40**	**21.0**



APPENDIX D

SUMMARY OF SEISMIC HAZARD ANALYSIS

Latitude, Longitude: 34.5101, -117.2762

Date	5/24/2023, 12:57:39 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.148	MCE_R ground motion. (for 0.2 second period)
S_1	0.442	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.195	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.796	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.041	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.493	MCE_G peak ground acceleration
F_{PGA}	1.107	Site amplification factor at PGA
PGA_M	0.546	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.148	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.226	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.442	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.48	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.493	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.936	Mapped value of the risk coefficient at short periods
C_{R1}	0.92	Mapped value of the risk coefficient at a period of 1 s
C_V	1.33	Vertical coefficient

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Determination of Site Class and Estimation of Shear Wave Velocity

Project: 12099.006 Mojave Narrows Restroom Addition

Depth (ft)	di, Layer Thick (ft)	Field Blow Counts, Ni Corrected for Cs and sampler type Blows per foot (bpf)		Average Ni (bpf)	Ni Hammer Corr:	di / Ni
		LB-1	LB-2		1.3	
5	7.5	8	9	9	11	0.68
10	5	8	8	8	10	0.48
15	5	9	7	8	10	0.48
20	5	33		33	43	0.12
25	5	52		52	68	0.07
30	5	75		75	98	0.05
35	5	20		20	26	0.19
40	5	21		21	27	0.18
45	5	32		32	42	0.12
50	7.5	48		48	62	0.12
60	10	30	*Assumed based on blowcount at 50'	30	39	0.26
70	10	30		30	39	0.26
80	10	30		30	39	0.26
90	10	30		30	39	0.26
100	5	30		30	39	0.13
Summation	100					3.65
					Navg = Sum(di) / Sum(di / Ni) =	**27**

Extract of ASCE 7-16 Table 20.3-1 Site Classification (2019 CBC 1613A.2.2):

Site Class	Soil Profile Name	Avg. N upper 100' from	to	Vs30 (ft/sec) from	to	Vs30 (m/s) from	to	Site Avg N	Interpolated vs30 (ft/s)
A	Hard Rock	-		5000	10000	1524	3048		
B	Rock	-		2500	5000	762	1524		
C	VD soil & soft rock	50.001	100	1200	2500	366	762		
D	Stiff Soil	**15**	**50**	600	1200	183	366	27	812
E	Soft Soil	0	14.999	0	600	0	183		
F		-	-			0	0		

SITE CLASS, Table 20.3-1: **D**

Estimation of Average Shear Wave Velocity in upper 100 ft (Vs30):

	ft/s	m/s
Approx. Vs30 (interpolation of Table 20.3-1) =	812	248
Approx. Vs30 sands (Imai and Tonouchi, 1982) =	990	302
Approx. Vs30 sands (Sykora and Stokoe, 1983) =	855	261
Approx. Vs30 (Maheswari, Boominathan, Dodagoudar, 2009) =	814	248

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 (u...

Latitude

Decimal degrees

34.51041738

Longitude

Decimal degrees, negative values for western longitudes

-117.2759934

Site Class

259 m/s (Site class D)

Spectral Period

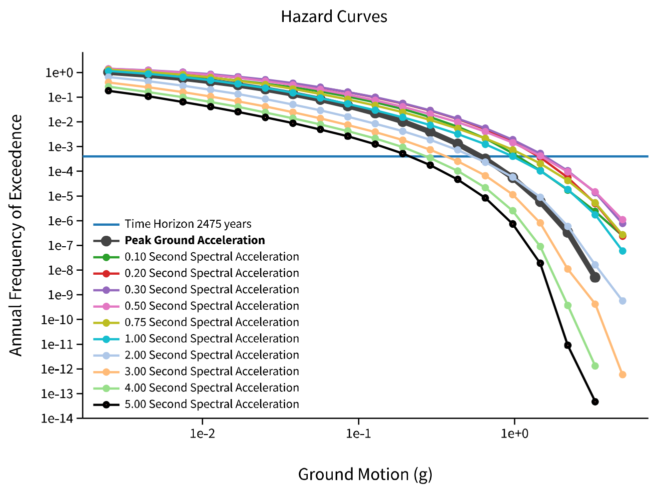
Peak Ground Acceleration

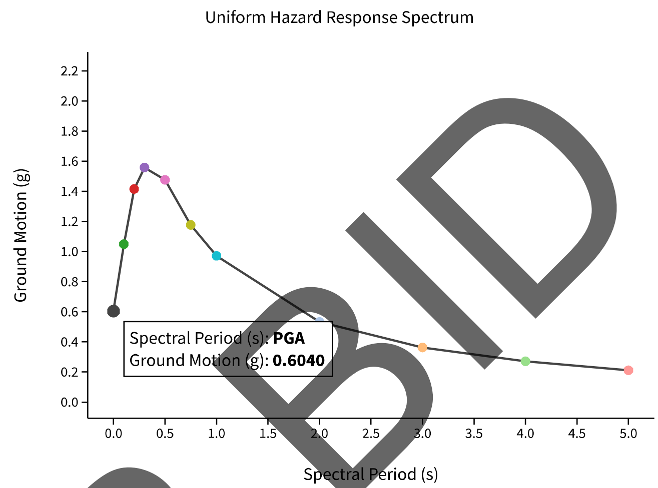
Time Horizon

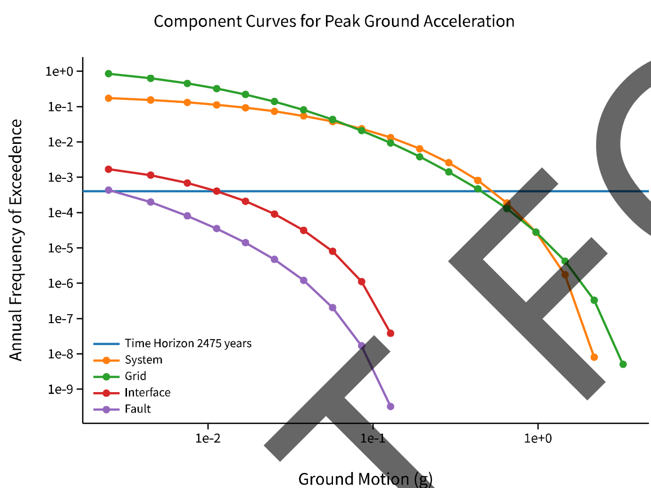
Return period in years

2475

Hazard Curve





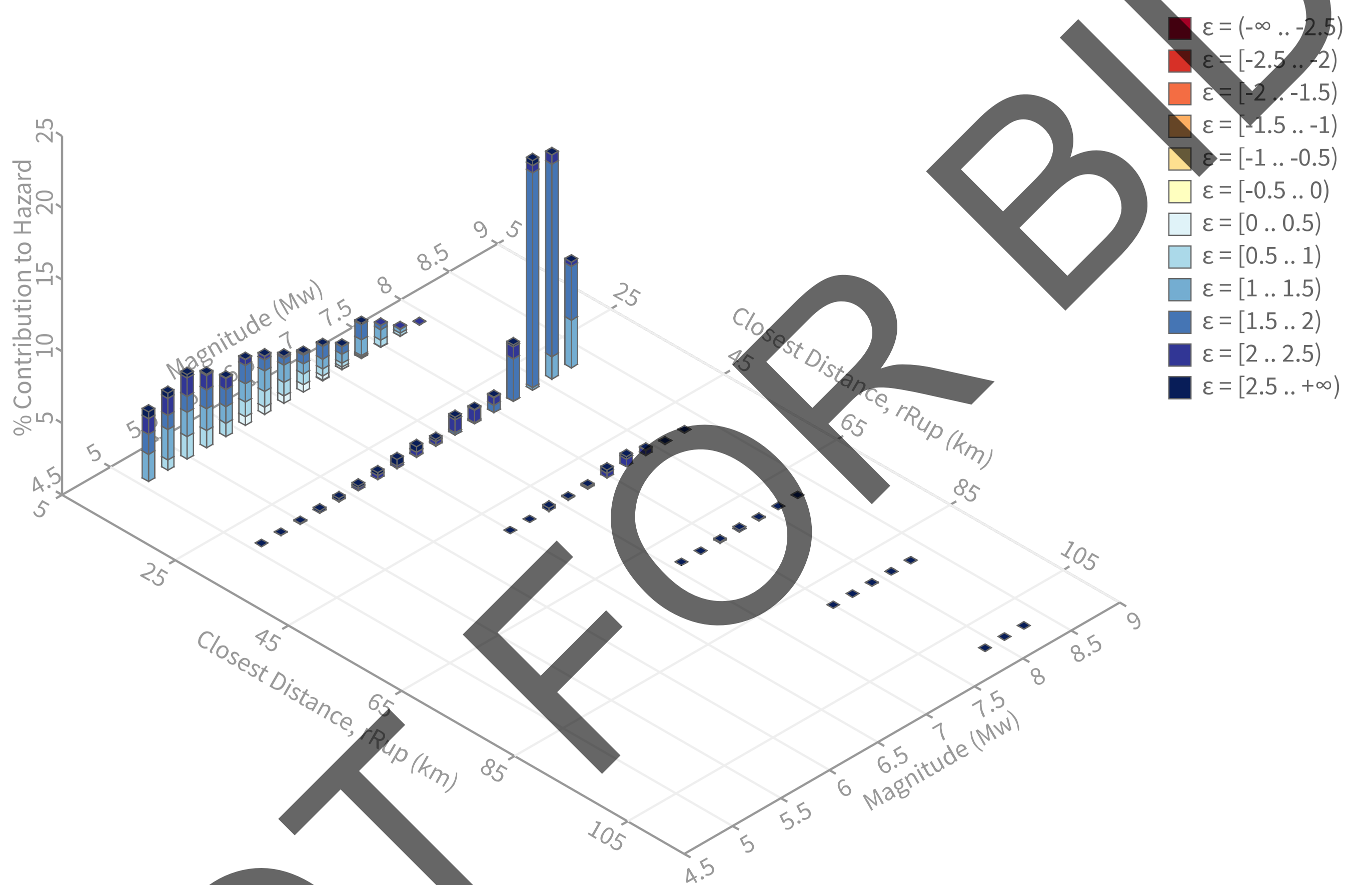


[View Raw Data]()

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.60396784 g

Recovered targets

Return period: 3000.8877 yrs
Exceedance rate: 0.00033323473 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.13 %

Mean (over all sources)

m: 6.98
r: 21.18 km
ε_0: 1.61 σ

Mode (largest m-r bin)

m: 7.91
r: 31.4 km
ε_0: 1.78 σ
Contribution: 15.99 %

Mode (largest m-r-ε_0 bin)

m: 7.91
r: 31.39 km
ε_0: 1.75 σ
Contribution: 14.96 %

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_FM32	System							28.49
San Andreas (San Bernardino N) [1]		31.12	7.99	1.72	117.456°W	34.273°N	212.07	20.20
North Frontal (West) [1]		13.98	7.27	1.30	117.169°W	34.422°N	134.91	1.86
Helendale-So Lockhart [7]		18.40	7.20	1.68	117.137°W	34.629°N	43.94	1.14
UC33brAvg_FM31	System							28.35
San Andreas (San Bernardino N) [1]		31.12	7.99	1.72	117.456°W	34.273°N	212.07	20.14
North Frontal (West) [1]		13.98	7.27	1.30	117.169°W	34.422°N	134.91	1.83
Helendale-So Lockhart [7]		18.40	7.21	1.68	117.137°W	34.629°N	43.94	1.15
UC33brAvg_FM31 (opt)	Grid							21.59
PointSourceFinite: -117.276, 34.533		5.54	5.73	1.00	117.276°W	34.533°N	0.00	4.58
PointSourceFinite: -117.276, 34.533		5.54	5.73	1.00	117.276°W	34.533°N	0.00	4.58
PointSourceFinite: -117.276, 34.578		8.50	5.82	1.42	117.276°W	34.578°N	0.00	1.77
PointSourceFinite: -117.276, 34.578		8.50	5.82	1.42	117.276°W	34.578°N	0.00	1.77
PointSourceFinite: -117.276, 34.623		12.28	5.91	1.82	117.276°W	34.623°N	0.00	1.01
PointSourceFinite: -117.276, 34.623		12.28	5.91	1.82	117.276°W	34.623°N	0.00	1.01
UC33brAvg_FM32 (opt)	Grid							21.57
PointSourceFinite: -117.276, 34.533		5.54	5.73	1.00	117.276°W	34.533°N	0.00	4.57
PointSourceFinite: -117.276, 34.533		5.54	5.73	1.00	117.276°W	34.533°N	0.00	4.57
PointSourceFinite: -117.276, 34.578		8.51	5.82	1.42	117.276°W	34.578°N	0.00	1.76
PointSourceFinite: -117.276, 34.578		8.51	5.82	1.42	117.276°W	34.578°N	0.00	1.76
PointSourceFinite: -117.276, 34.623		12.28	5.91	1.82	117.276°W	34.623°N	0.00	1.01
PointSourceFinite: -117.276, 34.623		12.28	5.91	1.82	117.276°W	34.623°N	0.00	1.01

Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Description: SBC Mojave Narrows Restroom GE; Case 1; PGAm 0.545; existing GW 9; No overex 0

Project No.: 12099.006

Apr 2023

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	9	9		0	2761	2752
LB-2	9	9		0	2760	2751
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)
0	0
4.6	46.6

General Parameters:

a_{max} = 0.55g

M_W = 7.9

MSF eq: 1

MSF = 0.88

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: SBC Mojave Narrows Restroom GE; Case 1; PGAm 0.545; existing GW 9; No overex 0

Project No.: 12099.006

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		95	120	12	2	1	7.8	300	13.9	21.7	0.238	300	0.35	0.40	**NonLiq**	21.7	0.06		0.03	2.5
LB-1	3.8 to 6.3	5	2.5		**47**	120	13	2	1	8.5	600	15.1	23.1	0.259	600	0.35	0.40	**NonLiq**	23.1	0.17		0.05	2.5
LB-1	6.3 to 8.8	7.5	2.5		5	120	13	2	1	8.5	900	14.4	**14.4**	0.154	900	0.35	0.40	**NonLiq**	14.4	0.53		0.16	2.5
LB-1	8.8 to 9.0	10	0.3		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**NonLiq**	13.6	0.92		0.03	2.3
LB-1	9.0 to 12.5	10	3.5		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**0.35**	13.6		2.06	0.87	2.3
LB-1	12.5 to 18.0	15	5.5		**1**	120	8	1	1.13	9.0	1426	13.0	**13.0**	0.141	1425.6	0.43	0.49	**0.29**	13.0		2.15	1.42	1.4
LB-1	18.0 to 23.0	21	5.0		10	120	51	2	1	33.2	1771	47.9	49.8	>Range	1771.2	0.48	0.55	**NonLiq**	49.8			0.00	0.0
LB-1	23.0 to 27.5	25	4.5		10	120	40	1	1.3	52.0	2002	70.6	73.0	>Range	2001.6	0.50	0.57	**NonLiq**	73.0			0.00	0.0
LB-1	27.5 to 32.5	30	5.0		5	120	58	1	1.3	75.4	2290	100.8	100.8	>Range	2289.6	0.52	0.59	**NonLiq**	100.8			0.00	0.0
LB-1	32.5 to 37.5	35	5.0		60	120	16	1	1.25	20.0	2578	25.2	35.3	>Range	2577.6	0.51	0.59	**NonLiq**	35.3			0.00	0.0
LB-1	37.5 to 42.5	40	5.0		**62**	120	17	1	1.25	21.3	2866	25.5	35.6	>Range	2865.6	0.50	0.58	**NonLiq**	35.6			0.00	0.0
LB-1	42.5 to 47.5	45	5.0		60	120	25	1	1.3	32.5	3154	37.0	49.4	>Range	3153.6	0.49	0.56	**NonLiq**	49.4			0.00	0.0
LB-1	47.5 to 52.0	50	4.5		60	120	37	1	1.3	48.1	3442	52.5	67.9	>Range	3441.6	0.47	0.54	**NonLiq**	67.9			0.00	0.0
LB-2	0 to 3.8	2.5	3.8		45	120	18	2	1	11.7	300	20.9	30.1	>Range	300	0.35	0.40	**NonLiq**	30.1	0.02		0.01	2.5
LB-2	3.8 to 6.3	5	2.5		45	120	15	2	1	9.8	600	17.4	25.9	0.310	600	0.35	0.40	**NonLiq**	25.9	0.14		0.04	2.5
LB-2	6.3 to 8.8	7.5	2.5		5	120	13	2	1	8.5	900	14.4	**14.4**	0.154	900	0.35	0.40	**NonLiq**	14.4	0.53		0.16	2.5
LB-2	8.8 to 9.0	10	0.3		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**NonLiq**	13.6	0.92		0.03	2.3
LB-2	9.0 to 12.5	10	3.5		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**0.35**	13.6		2.06	0.87	2.3
LB-2	12.5 to 17.0	15	4.5		5	120	6	1	1.1	6.6	1426	9.5	**9.5**	0.109	1425.6	0.43	0.49	**0.22**	9.5		2.67	1.44	1.4

Liquefaction Susceptibility Analysis: SPT Method

Leighton

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

Description: SBC Mojave Narrows Restroom GE; Case 2; PGAm 0.545; design GW 9; Overex. 5
Project No.: 12099.006
Apr 2023

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	9	9		5	2761	2752
LB-2	9	9		5	2760	2751
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Boring Location Coordinates Y (ft)
0	0
4.6	46.6

General Parameters:

a_{max} = 0.55g
M_W = 7.9
MSF eq: 1
MSF = 0.88
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: SBC Mojave Narrows Restroom GE; Case 2; PGAm 0.545; design GW 9; Overex. 5

Project No.: 12099.006

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	OX	95	120	50	1	1.3	65.0	300	116.0	144.2	>Range	300	0.35	0.40	**NonLiq**	144.2	0.00		0.00	2.5
LB-1	3.8 to 5.0	5	1.3	OX	**47**	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.35	0.40	**NonLiq**	144.2	0.00		0.00	2.5
LB-1	5.0 to 6.3	5	1.3		**47**	120	13	2	1	8.5	600	15.1	23.1	0.259	600	0.35	0.40	**NonLiq**	23.1	0.17		0.03	2.5
LB-1	6.3 to 8.8	7.5	2.5		5	120	13	2	1	8.5	900	14.4	**14.4**	0.154	900	0.35	0.40	**NonLiq**	14.4	0.53		0.16	2.5
LB-1	8.8 to 9.0	10	0.3		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**NonLiq**	13.6	0.92		0.03	2.3
LB-1	9.0 to 12.5	10	3.5		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**0.35**	13.6		2.06	0.87	2.3
LB-1	12.5 to 18.0	15	5.5		**1**	120	8	1	1.13	9.0	1426	13.0	**13.0**	0.141	1425.6	0.43	0.49	**0.29**	13.0		2.15	1.42	1.4
LB-1	18.0 to 23.0	21	5.0		10	120	51	2	1	33.2	1771	47.9	49.8	>Range	1771.2	0.48	0.55	**NonLiq**	49.8			0.00	0.0
LB-1	23.0 to 27.5	25	4.5		10	120	40	1	1.3	52.0	2002	70.6	73.0	>Range	2001.6	0.50	0.57	**NonLiq**	73.0			0.00	0.0
LB-1	27.5 to 32.5	30	5.0		5	120	58	1	1.3	75.4	2290	100.8	100.8	>Range	2289.6	0.52	0.59	**NonLiq**	100.8			0.00	0.0
LB-1	32.5 to 37.5	35	5.0		60	120	16	1	1.25	20.0	2578	25.2	35.3	>Range	2577.6	0.51	0.59	**NonLiq**	35.3			0.00	0.0
LB-1	37.5 to 42.5	40	5.0		**62**	120	17	1	1.25	21.3	2866	25.5	35.6	>Range	2865.6	0.50	0.58	**NonLiq**	35.6			0.00	0.0
LB-1	42.5 to 47.5	45	5.0		60	120	25	1	1.3	32.5	3154	37.0	49.4	>Range	3153.6	0.49	0.56	**NonLiq**	49.4			0.00	0.0
LB-1	47.5 to 52.0	50	4.5		60	120	37	1	1.3	48.1	3442	52.5	67.9	>Range	3441.6	0.47	0.54	**NonLiq**	67.9			0.00	0.0
LB-2	0 to 3.8	2.5	3.8	OX	45	120	50	1	1.3	65.0	300	116.0	144.2	>Range	300	0.35	0.40	**NonLiq**	144.2	0.00		0.00	2.5
LB-2	3.8 to 5.0	5	1.3	OX	45	120	50	1	1.3	65.0	600	116.0	144.2	>Range	600	0.35	0.40	**NonLiq**	144.2	0.00		0.00	2.5
LB-2	5.0 to 6.3	5	1.3		45	120	15	2	1	9.8	600	17.4	25.9	0.310	600	0.35	0.40	**NonLiq**	25.9	0.14		0.02	2.5
LB-2	6.3 to 8.8	7.5	2.5		5	120	13	2	1	8.5	900	14.4	**14.4**	0.154	900	0.35	0.40	**NonLiq**	14.4	0.53		0.16	2.5
LB-2	8.8 to 9.0	10	0.3		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**NonLiq**	13.6	0.92		0.03	2.3
LB-2	9.0 to 12.5	10	3.5		5	120	13	2	1	8.5	1138	13.6	**13.6**	0.147	1137.6	0.36	0.42	**0.35**	13.6		2.06	0.87	2.3
LB-2	12.5 to 17.0	15	4.5		5	120	6	1	1.1	6.6	1426	9.5	**9.5**	0.109	1425.6	0.43	0.49	**0.22**	9.5		2.67	1.44	1.4

Surface Manifestations of Liquefaction and Liquefaction Bearing Capacity Analysis

SBC Mojave Narrows Restroom GE; Case 2; PGAm 0.545; design GW 9; Overex. 5
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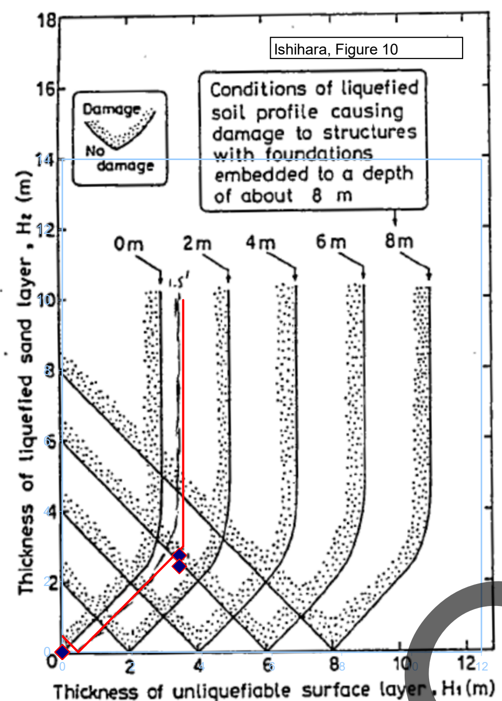
Boring No.	New Fill (raise grade) (ft)	Footing Depth (ft)	Ishihara, 1995, Surface Manifestisations of Liquefaction Analysis: Bot. Depth of Nonliq and Liq Layers Z1 (non) (ft)	Za (liq) (ft)	Zb (non) (ft)	Zc (liq) (ft)	Thickness H1 (ft)	Thickness H2 (ft)	Thickness H1 (m)	Thickness H2 (m)	Struct Damage/ Surface Manifestations? (Ishihara, 1995)	Amount of New Fill needed to mitigate (ft)	Or, Amount of Overex. needed to mitigate (ft)
LB-1	0	1.5	11.0	20.0	52.0		11.0	9.0	3.4	2.7	no	2	5
LB-2	0	1.5	11.0	19.0			11.0	8.0	3.4	2.4	no	2	5

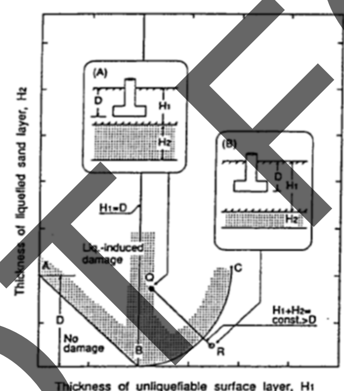
Karamitros et al., 2013, Liqufaction Bearing Capacity:

Boring No.	Assumed maximum Footing Width: Square ftg (ft)	Strip ftg (ft)	In order to achieve critical thickness of Non-liquefiable upper clay crust (where additional thickness does not further increase FSliq of bearing capacity): Amount of New Fill Needed (ft): square ftg	Strip ftg	Or, Amount of Overex. Needed (ft): square ftg	Strip ftg	Juang (2005) based on Iwasaki (1982), as presented in Tonkin & Taylor (2013), Liquefaction Potential Index (LPI): LPI = Sum[F1*W(z)*dz]	Risk of Liquefaction Damage Based on LPI
LB-1	6	2.5	0.0	0.0	0.0	0.0	0.0	Very Low
LB-2	6	2.5	0.0	0.0	0.0	0.0	0.0	Very Low

*** Considering 2-ft of Fill to Raise Grade for Proposed Grades**
*** Liquefiable Layer at 9 feet bgs + 2 feet of Fill = 11 feet**



Footing Depth= 1.5 ft
0.46 m



LPI range:	Liquefaction Risk:
LPI=0	Very low
0<LPI<=5	Low
5<LPI<=15	High
LPI>15	Very High

References:

Ishihara, K., 1995, Effects of At-Depth Liquefaction on Embedded Foundations During Earthquakes, Proceedings of 11th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Vol. 2, 1995.

Iwasaki, T., Arakawa, T., and Tokida, K., 1982, Simplifed Procedures for Assessing Soil Liquefaction During Earthquakes Proc. Conference on Soil Dynamics and Earthquake Engineering. Southampton, 925-939

Juang, C.H, Yang, S.H, Yuan, H., and Fang, S.Y., 2005, Liquefaction in the Chi-Chi earthquake – effect of fines and capping non-liquefiable layers Journal of the Japanese Geotechnical Society of Soils and Foundations, Vol. 45 No. 6 pp 89-101

Karamitros, Bouckovalas, Chaloulos, and Andrianopoulos, 2013, Numerical analysis of liquefaction-induced bearing capacity degradation of shallow foundations on a two-layered soil profile, Soil Dynamics and Earthquake Engineering, Vol 44.

Tonkin & Taylor Ltd, 2013, Liquefaction Vulnerability Study, Earthquake Commission, T&T Ref

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

LEIGHTON CONSULTING, INC.

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.