YELLOW

Geotechnical Engineering Exploration and Analysis

Proposed Public Storage Facility
SEQ Indian Street and Alessandro Boulevard
Moreno Valley, California

Prepared for:

Public Storage
Glendale, California

May 23, 2024
Project No. 2G-2404001

GILES
ENGINEERING ASSOCIATES, INC.

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ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

May 23, 2024

Public Storage, Inc.
701 Western Avenue
Glendale California 91201

Attention: Mr. Joe Tomlinson
Vice President of Construction

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Public Storage Facility
SEQ Indian Street and Alessandro Boulevard
Moreno Valley, California
Project No. 2G-2404001

Dear Mr. Tomlinson:

In accordance with your request and authorization, a *Geotechnical Engineering Exploration and Analysis* report has been prepared for the above-referenced project. Conclusions and recommendations developed from the exploration and analysis are discussed in the accompanying report.

We appreciate the opportunity to be of service on this project. If we may be of additional assistance, should geotechnical related problems occur or to provide construction observation and testing services, please do not hesitate to call at any time.

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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MORENO VALLEY, CALIFORNIA
PROJECT NO. 2G-2404001

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED PUBLIC STORAGE FACILITY
SEQ INDIAN STREET AND ALESSANDRO BOULEVARD
MORENO VALLEY, CALIFORNIA
PROJECT NO. 2G-2404001

EXECUTIVE SUMMARY OUTLINE

The executive summary is provided solely for the purpose of overview. Any party who relies on this report must read the full report. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report.

Subsurface Conditions

- Site Class designation D is recommended for seismic design considerations.
- Our review of the *Geologic Map of the Sunnymead/South ½ of Redlands Quadrangles (Dibblee, Jr., 2003)* indicates that the subject site is underlain by Quaternary Surficial Deposits (Qa) consisting of alluvial sand, gravel and clay of valley areas, covered with thick soil.
- Fill and possible fill soils were encountered in the test borings to depths of approximately zero to 3 feet below grade. Fill and native soils have very similar characteristics, and it was difficult to determine the exact geologic contact. Possible fill consisted of medium stiff to hard, moist, sandy silt, trace fine gravel, and medium dense silty sand, fine to medium grained, trace clay, with some mica.
- Beneath the possible fill, native soils were encountered within our test borings, which were generally loose to dense, silty sand, poorly graded sand with silt, and clayey sand, fine to medium grained, some coarse, some fine gravel, and stiff to hard sandy silt, silty clay, and sandy clay, fine sand, moist, some mica, and various amount of fine gravel and some mica.
- Groundwater was encountered at approximately 36 feet below ground surface in the Test Boring B-8 drilled to a maximum depth of 51.5 feet bgs.
- The expansion index (EI) test of the upper soils indicates a very low expansion potential.
- Based on our screening of corrosivity of the upper soils, the results from the minimum resistivity test generally indicate that the tested soils have a **moderately corrosive potential** when in contact with ferrous materials and chloride. The test results for pH indicated the **tested soil was neutral** and the near surface soils contain approximately 0.0207 percent of water-soluble sulfates; therefore, **no special sulfate resistant cement is considered necessary for concrete.**

Site Development

- It is our understanding that Public Storage will be developing this site by constructing one, 3-story building, with approximately 130,500 GSF with concrete slab-on-grade floor supported by spread footings or a mat foundation, and paved driveways and parking lot areas. The proposed building will have no basement level.
- New Building: Due to the variable and low strength characteristics of the near surface onsite soils and the estimated seismic-induced seismic dry sand settlement, and to develop uniformity of support, it is recommended that the soils within the proposed new building areas and an appropriate lateral distance beyond (5 feet minimum) be over-excavated to a depth of at least 5 feet below the proposed grade or 3 feet below bottom of foundations and floor slab, whichever is deeper. For the planned subgrade, the existing soils should be proofrolled to remove any unstable materials and the surface compacted to an in-place density of at least 90% of its maximum dry density per ASTM D-1557. The existing soils are considered suitable for foundation and floor support with the recommended 3-foot structural fill layer for the foundations only, and the existing

soils are also considered suitable for pavement support with recommended proofroll and geotechnical inspection/testing. The soils exposed after cutting to the structural fill subgrade should be examined by the geotechnical engineer to document that the soils are suitable for building support. Prior to placement of fill, the exposed surfaces approved for fill placement should be scarified to a depth of at least 6 to 8 inches, moisture conditioned above optimum moisture content and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557).

Building Foundation

- The proposed structure may be supported by a shallow spread footing foundation system or a mat foundation for a maximum, net allowable soil bearing pressure of 3,500 pounds per square foot (psf) underlain by a minimum 3-foot structural compacted fill layer. Structural loads supported by a mat foundation may be designed for a maximum modulus of subgrade reaction (Ks) of 75 pounds per square inch per inch (psfi/in.).
- Foundation reinforcement should be determined by the structural engineer.

Building Floor Slab

- It is recommended that an on-grade slab be a minimum 4-inch-thick slab-on-grade or turned-down slab, underlain by a minimum 4-inch-thick granular base supported on a properly prepared subgrade consisting of a minimum 1-foot structural fill layer.
- A minimum 15-mil vapor barrier is recommended to be directly below the floor slab or base course where required to protect moisture sensitive floor coverings.

New Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 4 or 6 inches of base course in parking stall and drive lane areas, respectively.
- Portland Cement Concrete: 6 inches in thickness underlain by 4 inches of base course in high stress areas such as entrance/exit aprons, drive-thru lane and the trash enclosure-loading zone.

Construction Considerations

- Due to anticipated on-site soils, loose to medium dense sandy soils, shallow excavations, up to 5 feet in vertical height, may be adequately sloped for bank stability, where sufficient space is available, temporarily unsurcharged embankments could be sloped back at a 2:1 (h:v) slope gradient. Deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

YELLOW – This site has been given a yellow designation due to potential increased costs associated with the minimum over-excavation and placement of a compacted fill below the footings during construction for the recommended spread footings/mat foundation due to the estimated dry seismic settlement.

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report.

Geotechnical-related recommendations for design and construction of the foundation and ground-bearing floor slab for the proposed buildings are provided in this report. Geotechnical-related recommendations are also provided for the proposed driveways. Site preparation recommendations are also given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include the weather before and during construction, the water table at the time of construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development.

2.0 SITES AND PROJECT DESCRIPTION

2.1 Site Description

The subject property is at the southeast quadrangle of Indian Street and Alessandro Boulevard, in the city of Moreno Valley, Riverside County, California. Based on internet research, the subject property has a history of being an undeveloped vacant lot, covered with soils and some vegetation. The site is bounded on the north by an existing storage facility, on the east by residential structures, on the south a gas station, commercial buildings and Alessandro Blvd further south, and on the west by Indian Street. The neighboring development consists of a mixture of residential structures and commercial structures.

Based on Google Earth website, the site gently slopes with an approximately 4-ft elevation difference across the site, sloping toward the south (to approximate El. 1579) from the north area (approximately El. 1583).

2.2 Proposed Project Description

It is our understanding that Public Storage will be developing this site by constructing one, 3-story building, with approximately 130,500 GSF with concrete slab-on-grade floor supported by spread footings or a mat foundation, and paved driveways and parking lot areas. The proposed building will have no basement level. A Conceptual Site Plan was provided to Giles. Structural loads were not available at the time this report was prepared, but we have estimated maximum building loads, based on a 10-foot by 10-foot column spacing, to be about 75 to 100 kips for interior columns and 2 to 3 kips per foot for walls. When design loading conditions are final, we should be notified to re-evaluate the conclusions and recommendations contained in this report. The structure is anticipated to be concrete, masonry, and steel.

It is anticipated that the finished floor elevation of the new building will be near or at existing grade. The finished floor elevation is anticipated to be approximately at El. 1583. Therefore, site grading is anticipated to include minor cut and fills to establish the necessary site grade to accommodate the assumed floor elevation, exclusive of site preparation or over-excavation requirements necessary to create a stable site suited for the proposed development.

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of 15 test borings (B-1 through B-15) to depths of approximately 10 to 51.5 feet below existing ground surface utilizing a truck rig with hollow-stem auger drilling equipment. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

Standard split-spoon tests (SS), also called Standard Penetration Test (SPT), were performed at selected depth intervals in accordance with the American Society for Testing Materials (ASTM) Standard Procedure D 1586. This method consists of mechanically driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The number of blows required to drive the standard split-spoon sampler for the last 12 of the 18 inches was identified as the uncorrected standard penetration resistance (N). Disturbed soil samples from the unlined standard split-spoon samplers were placed in plastic bags and transported to our laboratory for testing. A representative bulk sample, consisting of composite soil materials from the upper soils, was obtained from all borings.

Our subsurface exploration included also the collection of relatively undisturbed soil samples of subsurface soil materials at selected depth intervals from the test borings for laboratory testing purposes. Relatively undisturbed samples were collected (per ASTM D 3550) using a 3-inch outside diameter, modified California split-spoon soil sampler (CS) lined with 1-inch-high brass rings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

At the conclusion of drilling activities, each borehole was backfilled. Even with this service, however, it is important to note that some boreholes backfill settlement or expansion can and will occur over time. This settlement/expansion can create a hazard and should be carefully monitored by the client and/or property owner. The settlement/expansion can lead to the formation of a “trip joint” representing a threat of injury to persons or animals utilizing or accessing the subject property. Giles has not included a cost for monitoring borehole settlement/expansion after the initial drilling activities and will not be performing this service.

3.2 Subsurface Conditions

The subsurface conditions as subsequently described have been simplified somewhat for ease of report interpretation. A more detailed description of the subsurface conditions at the test boring locations is provided by the logs of the test borings enclosed in Appendix A of this report.

Site Geology

Our review of the *Geologic Map of the Sunnymead/South ½ of Redlands Quadrangles (Dibblee, Jr., 2003)* indicates that the subject site is underlain by Quaternary Surficial Deposits (Qa) consisting of alluvial sand, gravel and clay of valley areas, covered with thick soil.

Soil

Fill and possible fill soils were encountered in the test borings to depths of approximately zero to 3 feet below grade. Fill and native soils have very similar characteristics, and it was difficult to determine the exact geologic contact. Possible fill consisted of medium stiff to hard, moist, sandy silt, trace fine gravel, and medium dense silty sand, fine to medium grained, trace clay, with some mica.

Beneath the possible fill, native soils were encountered within our test borings, which were generally loose to dense, silty sand, poorly graded sand with silt, and clayey sand, fine to medium grained, some coarse, some fine gravel, and stiff to hard sandy silt, silty clay, and sandy clay, fine sand, moist, some mica, and various amount of fine gravel.

Groundwater

Groundwater was encountered at approximately 36 feet below ground in the deep boring (B-8) during our field exploration. Based on a review of GeoTracker website, groundwater was found at 35 feet below grade from a well located approximately 400 feet southeast of our subject site. The historic groundwater level has not been mapped in this area.

Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

3.3 Percolation Testing

It is our understanding that an on-site below grade storm water infiltration system is being considered for the subject site. Therefore, two percolation tests were performed to assess the infiltration characteristics of the site soils.

The percolation testing consisted of drilling an 8-inch-diameter hole using a hollow-stem auger, installing a 2-inch-diameter slotted pvc casing with a solid end cap and then surrounding the casing with a granular filter pack. The test holes (B-10 and B-12) were then pre-soaked to a minimum depth of 1 foot above the bottom of the boring. After pre-soaking, test water was added to the casing and refilled after each consecutive percolation test reading. The drop in water level over time is the percolation rate at the test location. The percolation test procedure outlined in the Riverside County Department of Environmental Health was used as a guide in our testing. A summary Table of the test results of the percolation tests is provided below.

The drop in water level over time is the pre-adjusted percolation rate at the test location. The percolation rate was reduced to account for the discharge of water from the sides and bottom of the boring. The Porchet Method, noted below, was used to calculate the design infiltration rate.

Tested Infiltration Rate = $\Delta H\ (60r) / \Delta t\ (r + 2H_{avg})$

Where: r is the radius of the test hole (in)
ΔH is the change in height over the time interval (in)
Δt is the time interval (min)
Havg is the average head height over the time interval.

The results obtained from our percolation testing are summarized below. The infiltration rates noted below have not been reduced to account for a factor of safety.

Test Number	Test Depth (feet)	Design Infiltration Rate (in/hr)	Soil Type
B-10	10	4.38	(SP) – Poorly-graded Sand
B-12	10	2.83	(SP) – Poorly-graded Sand

It should be noted that the infiltration rate of the on-site soils represents a specific area and depth tested and may fluctuate throughout other parts of the site.

4.0 LABORATORY TESTING

Several laboratory tests were performed on selected samples considered representative of those encountered in order to evaluate the engineering properties of the on-site soils. The following is a brief description of our laboratory test results.

In Situ Moisture and Density

Tests were performed on select samples from the test borings to determine the subsoils dry density and natural moisture contents in accordance with Test Method ASTM D 2216. The results of these tests are included in the Test Boring Logs enclosed in Appendix A.

Expansive Potential

To evaluate the expansion potential of the near surface soils encountered during our subsurface exploration, a representative bulk soil sample collected from the upper soils from the test borings was subjected to Expansion Index (EI) testing in accordance with Test Method ASTM D 4829. The result of our expansion index (EI) test indicates that the near surface sample has a very low expansion potential (EI < 5).

Sieve Analysis

Sieve Analyses (Passing No. 200 Sieve) were performed on selected samples to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 1140. The results of the Passing No. 200 Sieve tests are presented in Test Boring Logs in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

A representative bulk sample of the near surface soils which may contact shallow buried utilities and structural concrete was used to perform to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water-soluble sulfate which could result in chemical attack of cement. The following table presents the results of our laboratory testing.

Parameter	**Bulk Sample 0 to 5 feet**
pH	7.1
Chloride	223 ppm
Sulfate	0.0207%
Resistivity	8,100 ohm-cm

The chloride content of near-surface soils was determined for a selected sample in accordance with California Test Method No. 422. The results of this test indicated that **tested on-site soils have a moderate exposure to chloride.**

The results of limited testing of soil pH and minimum resistivity were determined in accordance with California Test Method No. 643. The test results for pH indicated the **tested soil was neutral**. The results from the minimum resistivity test generally indicate that the tested soils have a **moderately corrosive potential** when in contact with ferrous materials. Therefore, special protection for underground cast iron pipe or ductile pipe may be warranted depending on the actual materials in contact with the pipe. We recommend that a corrosion engineer review these results in order to provide specific recommendations for corrosion protection as well as appropriate recommendations for other types of buried metal structures.

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the concentrations present of water-soluble sulfate which could result in chemical attack of cement. Our laboratory test data indicated that **near surface soils contain approximately 0.0207 percent of water-soluble sulfates**. Based on Section 1904.1 of the California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318 a negligible exposure to sulfate can be expected for concrete placed in contact with the tested on-site soils. **No special sulfate resistant cement is considered necessary for concrete** which will be in contact with the tested on-site soils.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 Liquefaction

According to the City of Moreno Valley Safety Elements, Map S-2, Liquefaction Hazards, the site is located within a low to moderate liquefaction susceptibility zone. During our field exploration, drilled to a maximum depth of 51.5 feet bgs, groundwater was encountered at approximately 36 feet below ground surface (bgs). Based GeoTracker, groundwater was found be about 35 feet bgs from a nearby well. Silt and Clay soil types interbedded with sandy layers were found at those depths based on our recent field exploration. Therefore, based on encountered groundwater level and soil type a liquefaction analysis for this site was performed.

5.2 Active Fault Zones

Based on the City of Moreno Valley, Riverside County Seismic and Geologic Hazards, dated 2019, Map S-1, Fault Zones and the Special Studies Zones for the Sunnymead Quadrangle (CGS, 1974), the site is not located within any active faults zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Conditions imposed by the proposed development have been evaluated on the basis of the assumed floor elevation and engineering characteristics of the subsurface materials encountered during our subsurface investigation and their anticipated behavior both during and after construction. Conclusions and recommendations presented for the design of building foundations, floor slab, and pavement along with site preparation recommendations and construction considerations are discussed in the following sections of this report.

From a soils engineering point of view, the subject property is considered geotechnically suitable for the proposed new development provided the following recommendations are incorporated in the design and construction of the project.

We recommend that Giles Engineering Associates, Inc. be involved in the review of the grading and foundation plans for the site to ensure our recommendations are interpreted correctly. Based on the results of our review, modifications to our recommendations or the plans may be warranted.

Effect of Proposed Grading and Construction on Adjacent Property

It is our opinion that the proposed construction and grading will be safe against geotechnical hazards from landslides, settlement, or slippage and the proposed work will not adversely affect the geologic stability of the adjacent property provided grading and construction are performed in compliance with the local city code and in accordance with the recommendations presented herein.

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

The site is not located within any active fault zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the *California Building Code (CBC)* and applicable local codes. In accordance with *ASCE 7*, Chapter 20, a Site Classification D is recommended for this site based upon the mapped geological features of the site also verified by test borings.

According to the maps of known active fault near-source zones to be used with the CBC, the San Jacinto fault and its branches are the closest known active faults and located about 4.9 and 6.8 miles from our subject site. The San Jacinto fault would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 7.88 (Hanks).

The proposed structure should be designed in accordance with the current version of the *California Building Code (CBC)*, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures ASCE 7*, and applicable local codes. The following values are determined by using the SEAOC/OSHPD Seismic Design Map Tool based upon the *CBC 2022* and *ASCE 7-16*.

CBC 2022, Earthquake Loads	
Site Class Definition (Table 20.3-1 from ASCE 7-16)	D
Mapped Spectral Response Acceleration Parameter, S_s (for 0.2 second)	1.579
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.613
Site Coefficient, F_a short period	1.00
Site Coefficient, F_v 1-second period	1.70
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS}	1.579
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1}	1.042
Design Spectral Response Acceleration Parameter, S_{DS}	1.052
Design Spectral Response Acceleration Parameter, S_{D1}	0.695
MCEG Peak Ground Acceleration adjusted for site class effects, PGA_M	0.737

According to Section 11.4.8 of ASCE 7-16 for structural engineering considerations, a ground motion hazard analysis is required and should be performed in accordance with Section 21.2 for structures on Site Class D with S_1 greater than or equal to 0.2. However, as an exception to performing the ground motion hazard analysis, the value of the Seismic Response Coefficient (Cs) must be determined by Equation (12.8-2) for values of the fundamental period of the building (T) ≤ 1.5Ts, and taken as 1.5 times the value computed in accordance with either Equation (12.8-3) for T_L ≥ 1.5Ts, or Equation (12.8-4) for T > T_L.

Liquefaction

A Seismic Hazard Evaluation Report for the area where the subject site is located has not yet been prepared by the CGS (California Geological Survey). However, based on a review of the Moreno Valley Safety Elements, the site is located within a low to moderate liquefaction potential. Based on a review of groundwater information on the State of California GeoTracker website, the depth to groundwater in the vicinity of the site is approximately 35 feet below grade. Accordingly, a detailed liquefaction analysis was considered necessary and performed for this site.

Liquefaction analysis was performed based on the current California Building Code (2022 CBC) and was performed utilizing the Standard Penetration Test (SPT) data from Test Boring B-8. The NCEER method (1998) was applied as presented by Youd, et.al (2001). The peak ground acceleration (PGA) and modal magnitude were obtained from the USGS Seismic Ground Motion Design Parameter Tool and USGS Probabilistic Seismic Hazards Deaggregation Tool, respectively. As per ASCE 7-16 guidelines, the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for the site effects (PGA_M) equals to 0.737g and a predominant mean earthquake magnitude of 7.38 was used in our analysis.

The on-site fine-grained soils were evaluated to determine susceptibility to liquefaction during ground shaking in accordance with the criteria outlined within SP117A and the parameters and soil profile mentioned above. Based on our field exploration, laboratory test results, and soil type, the results of our liquefaction evaluation indicate that the ground settlement resulting from the design-level earthquake will be negligible. The results of our liquefaction analysis are provided as Plate A-1 of Appendix A.

Seismic-Induced Settlement

Seismic-induced settlement is often caused by loose to medium dense granular soils densified during ground shaking. To evaluate the site-specific potential for seismic-induced settlement above the groundwater level (assumed at 35 feet), we used the same parameters used in the liquefaction analysis based on the 2022 CBC and ASCE 7-16. Based on the results of our analysis, the seismic-induced settlement is estimated to be on the order of approximately 2.15 inches. However, as a result of over-excavating the upper 5 feet or 3-ft below bottom of the footings, the dry settlement is reduced to 0.40 inches. Therefore, differential settlement may be assumed to be negligible. The results of our analysis performed at Test Boring B-8 are presented as Plate A-2 of Appendix A.

Liquefaction-Induced Lateral Spreading

Lateral spreading of the ground surface during a seismic activity usually occurs along the weak shear zones within a liquefiable soil layer and has been observed to generally take place toward a free face (i.e. retaining wall, slope or channel) and to lesser extend on ground surfaces with a very gentle slope. Due to absence of any slope or channel within or near the subject site, the potential for lateral spread occurring within the site is considered to be very low.

6.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations and the results of our laboratory testing.

Site Clearing and Preparation

Clearing and demolition operations should include the removal of all landscape vegetation and any existing structural features, within the area of the proposed new building and site improvements. All soils disturbed by grading operations should be removed and/or compacted to provide a competent subgrade, as determined by the project geotechnical engineer.

Should any unusual soil conditions or subsurface structures be encountered during clearing/demolition operations or during grading, they should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Existing Utilities

All existing utilities should be located. Utilities that are not reused should be capped off and removed or properly abandoned in-place in accordance with city codes and ordinances. The excavations made for removed utilities that are in the influence zone of new construction are recommended to be backfilled with structural compacted fill. Underground utilities, which are to be reused or abandoned in-place, are recommended to be evaluated by the structural engineer and utility backfill is recommended to be evaluated by the geotechnical engineer, to determine their potential effect on the new development. If any existing utilities are to be preserved, construction operations must be carefully performed so as not to disturb or damage the existing utility.

Building Area

Due to the variable and low strength characteristics of the near surface onsite soils and the estimated seismic-induced seismic settlement, and to develop uniformity of support, it is recommended that the soils within the proposed new building areas and an appropriate lateral distance beyond (5 feet minimum) be over-excavated to a depth of at least 5 feet below the proposed grade or 3 feet below bottom of foundations and floor slab, whichever is deeper. For the planned subgrade, the existing soils should be proofrolled to remove any unstable materials and the surface compacted to an in-place density of at least 90% of its maximum dry density per ASTM D-1557. The existing soils are considered suitable for foundation and floor support with the recommended 3-foot structural fill layer, and soils are also suitable for pavement support with recommended proofroll and geotechnical inspection/testing. The soils exposed after cutting to the structural fill subgrade should be examined by the geotechnical engineer to document that the soils are suitable for building support. Prior to placement of fill, the exposed surfaces approved for fill placement should be scarified to a depth of at least 6 to 8 inches, moisture conditioned to above the soil's optimum moisture content, and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557).

Positive drainage devices such as sloped concrete flatwork, earth swales, and sheet flow gradients in landscape, setback, and easement areas should be designed for the site. The drainage system should drain to a suitable discharge area. The purpose of this drainage system is to reduce water infiltration into the subgrade soils and to direct water away from buildings and site improvements.

All utility trenches backfill should be placed in lifts no greater than 8 inches in thickness, moisture conditioned above optimum moisture content and then compacted to a minimum of 90 percent of the soil's maximum density near the optimum moisture content. A representative of the project geotechnical engineer should observe and test the backfills to document adequacy of compaction.

Proofroll and Compact Subgrade

Following site clearing, removal or re-compaction of disturbed soils and lowering of site grades where necessary for the 3-foot structural fill layer in the building area, the subgrades within the proposed building, pavement and drive areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade, or stabilized in place. Any unsuitable materials discovered should be removed and backfilled with structural fill. Following proofrolling and completion of any necessary over-excavation, the subgrades in the building, parking lot and drive areas should be scarified to a depth of 6 to 8 inches, moisture conditioned above the soil's optimum moisture content, and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable low-expansive structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

The Guide Specifications included in Appendix D (Modified Proctor) of this report are recommended to be used, at a minimum, as an aid in developing the project specifications. The floor slab subgrade may need to be recompacted prior to slab construction due to weather and equipment traffic effects on the previously compacted soil.

Reuse of On-site Soil

On-site material may be reused as structural compacted fill (if needed) within any new construction area provided they do not contain oversized materials and significant quantities of organic matter or other deleterious materials. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering-controlled conditions.

Subgrade Protection

The near surface soils that are expected to comprise the subgrade are sensitive to water and disturbance from construction activities. Unstable soil conditions will develop if the soils are exposed to moisture increases or are disturbed (rutted) by construction traffic. If unstable soil conditions occur, recommendations for stabilization should be provided by the geotechnical engineer at the time of grading/construction based on the conditions encountered. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade. The degree of subgrade instability and associated remedial construction is dependent, in part, upon precautions taken by the contractor to protect the subgrade during site development.

Silt fences or other appropriate erosion control devices should be installed in accordance with local, state and federal requirements at the perimeter of the development areas to control sediment from erosion. Since silt fences or other erosion control measures are temporary structures, careful and continuous monitoring and periodic maintenance to remove accumulated soil and/or replacement should be anticipated.

Fill Placement

All fill should be placed in 8-inch-thick maximum loose lift, moisture conditioned above the soil's optimum moisture content and then compacted to at least 90 percent, or 95 percent in the upper 12 inches of pavement subgrade of the Modified Proctor maximum density. A representative of the project geotechnical consultant should be present on-site during grading operations to document proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Import Structural Fill

Any soils imported to the site (if required) for use as structural fill should consist of very low (EI less than 20) expansive soils. Materials designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure, soluble sulfate content and corrosivity and pavement support characteristics.

6.3 Construction Considerations

Construction Dewatering

Groundwater was encountered at approximately 36 feet bgs during our subsurface exploration to the maximum depth explored (51.5 feet). However, the site may be susceptible to a shallower perched water table due to seasonal precipitation and runoff characteristics of the site. Conventional filtered sump pumps placed in excavations are expected to be suitable for dewatering should any excess water conditions be observed.

Soil Excavation

Some localized slope stability problems may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Due to anticipated on-site soils, loose to medium dense sandy soils, shallow excavations, up to 5 feet in vertical height, may be adequately sloped for bank stability, where sufficient space is available, temporarily unsurcharged embankments could be sloped back at a 2:1 (h:v) slope gradient. Deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

6.4 Foundation Recommendations

Vertical Load Capacity – Shallow Foundation

Upon completion of the recommended building pad preparation, it is our opinion the proposed structures may be supported by a shallow foundation system and/or mat slab. Foundations underlain by a minimum 3-foot structural fill layer may be designed for a maximum, net, allowable soil-bearing pressure of 3,500 pounds per square foot (psf). Structural loads supported by a mat foundation may be designed for a maximum modulus of subgrade reaction (Ks) of 75 pounds per square inch per inch (psfi/in.). Minimum foundation widths for walls and columns should be 18 and 24 inches, respectively, for bearing considerations, regardless of actual soil pressure. The maximum bearing value applies to combined dead and sustained live loads. This allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads. A one-third increase in the passive pressure value may be used for short duration wind or seismic loads.

A coefficient of friction of 0.35 may be used with dead load forces for footings placed on newly placed compacted fill soil. An allowable passive earth pressure of 400 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against newly placed structural fill. The maximum recommended allowable passive pressure is 3,500 psf.

Foundation Embedment

The California Building Code (CBC) requires a minimum 12-inch foundation embedment depth. However, it is recommended that exterior foundations extend at least 18 inches below the adjacent exterior grade for bearing capacity and to provide greater protection of the moisture sensitive bearing soils. Interior footings may be supported at nominal depth below the floor. All footings must be protected against weather and water damage during and after construction and must be supported within suitable bearing materials.

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than 1 and ½ inch, respectively. The estimated differential movement is anticipated to result in an angular distortion of about 0.002 inches per inch on the basis of a minimum clear span of 20 feet for a standard foundation; and 0.001 in./in. on the basis of a minimum clear span of 40 feet for a mat foundation and 0.001 in/in.

Reinforcing

The determination of the actual quantity of steel reinforcing and dimensions should be performed by the project structural engineer.

Bearing Material Criteria

Soil suitable to serve as the structural fill subgrade should exhibit at least a loose relative density (average N value of at least 10) for non-cohesive soils, and an unconfined compressive strength of 1.75 tsf for cohesive soils, for the recommended 3,500 psf allowable soil bearing pressure. For design and construction estimating purposes, suitable structural fill subgrade soils are expected to be encountered at the recommended 3-foot structural fill depths following the recommended site preparation activities. However, field testing by the Geotechnical Engineer within the structural fill subgrade and the structural fill soils is recommended to document that the foundation support soils possess the minimum strength parameters noted above. Testing may consist of Dynamic Cone Penetration tests (per ASTM Special Publication 399), nuclear gauge, sand cone test, other tests as deemed suitable by the Geotechnical Engineer. If unsuitable bearing soils are encountered, they should be recompacted in-place, if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade.

6.5 Floor Slab Recommendations

Subgrade

The floor slab subgrade should be prepared in accordance with the appropriate recommendations presented in the Site Development Recommendations section of this report including a minimum 2-foot structural fill layer. Foundation, utility trenches and other below-slab excavations should be backfilled with structural compacted fill in accordance with the project specifications.

Design (Conventional Slab-on-Grade)

The floor of the proposed building is recommended to be designed as a conventional slab-on-grade where the floor slab is independent of the foundations. The at-grade floor slab may be designed based on a maximum modulus of subgrade reaction (k_s) of 175 pounds per cubic inch (pci), supported on a properly prepared subgrade consisting of a minimum 3-foot structural fill layer. If desired, the floor slab may be poured monolithically with foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. The slab is recommended to be a minimum of 4 inches in thickness. A qualified structural engineer should perform the actual design of the slab to ensure proper thickness and reinforcing.

The floor slab is recommended to be underlain by a 4-inch-thick layer of granular base, which is not included as a portion of the recommended structural fill layer recommended in Section 6.2 Building Area. A 15-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.) and control moisture through the floor slab. It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture. The vapor retarder is recommended to be in accordance with ASTM E 1745, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch-thick layer of sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Settlement

Post-construction total and differential movements of the floor slab designed and constructed in accordance with the recommendations provided in this report are estimated to be less than ½ and ⅓ inch, respectively. Movements on the order of those estimated for foundations should be expected when the foundation and floor slab are structurally connected or constructed monolithically. The estimated differential movement is anticipated to occur across the short dimension of the structure.

6.6 New Pavement

The following recommendations for the new pavement are intended for vehicular traffic associated with the restaurant development within the subject property.

New Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction is expected to consist of existing on-site soil that exhibit a very low expansion potential. An R-value of 30 has been assumed in the preparation of the pavement design. It should, however, be recognized that the City of Moreno Valley may require a specific R-value test to verify the use of the following design. It is recommended that this testing, if required, be conducted following completion of rough grading in the proposed pavement areas so that the R-value test results are indicative of the actual pavement subgrade soils. Alternatively, a minimum code pavement section may be required if a specific R-value test is not performed. To use this R-value, all fill added to the pavement subgrade must have pavement support characteristics at least equivalent to the existing soils and must be placed and compacted in accordance with the project specifications.

Asphalt Pavements

The following table presents recommended thicknesses for a new flexible pavement structure consisting of asphaltic concrete over a granular base, along with the appropriate CALTRANS specifications for proper materials and placement procedures. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas. However, care must be used so that truck traffic is excluded from areas where the thinner pavement section is used, since premature pavement distress may occur. In the event that heavy vehicle traffic cannot be excluded from the specific areas, the pavement section recommended for drive lanes should be used throughout the parking lot.

ASPHALT PAVEMENTS			
Materials	**Thickness (inches)**		**CALTRANS Specifications**
	Parking Stalls (TI=4.0)	**Drive Lanes (TI=5.0)**	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	4	6	Section 26, Class 2 (R-value at least 78)

NOTES:
(a) Compaction to density between 95 and 100 percent of the 50-Blow Marshall Density
(b) The surface and binder course may be combined as a single layer placed in one lift if similar materials are utilized.

Pavement recommendations are based upon CALTRANS design parameters for a twenty-year design period and assume proper drainage and construction monitoring. It is, therefore, recommended that the geotechnical engineer monitors and tests subgrade preparation, and that the subgrade be evaluated immediately before pavement construction.

Portland Concrete Pavements

Portland Cement Concrete pavements are recommended in areas where traffic is concentrated such as the entrance/exit aprons as well as areas subjected to heavy loads such as the trash enclosure loading zone. The preparation of the subgrade soils within concrete pavement areas should be performed as previously described in this report. Portland Cement Concrete pavements in high stress areas are recommended to be at least 6 inches thick containing No. 3 bars at 18-inch on-center both ways placed at mid-height. The pavement should be constructed in accordance with Section 40 of the CALTRANS Standard Specifications. A minimum 4-inch-thick layer of base course (CALTRANS Class 2) is recommended below the concrete pavement. This base course should be compacted to at least 95% of the material's maximum dry density.

The maximum joint spacing within all of the Portland Cement Concrete pavements is recommended to be 15 feet or less to control shrinkage cracking. Load transfer reinforcing is recommended at construction joints perpendicular to traffic flow if construction joints are not properly keyed. In this event, ¾-inch diameter smooth dowel bars, 18 inches in length placed at 12 inches on-center are recommended where joints are perpendicular to the anticipated traffic flow. Expansion joints are recommended only where the pavement abuts fixed objects such as light standard foundations. Tie bars are recommended at the first joint within the perimeter of the concrete pavement area. Tie bars are recommended to be No. 4 bars at 42-inch on-center spacings and at least 48 inches in length.

General Considerations

Pavement recommendations assume proper drainage and construction monitoring and are based on traffic loads as indicated previously. Pavement designs are based on either PCA or CALTRANS design parameters for twenty (20) year design period. However, these designs are also based on a routine pavement maintenance program and significant asphalt concrete pavement rehabilitation after about 8 to 10 years, in order to obtain a reasonable pavement service life. Due to the presence of variable strength characteristics of the near surface on-site soils, some increased pavement maintenance should be expected.

6.7 Recommended Construction Materials Testing Services

The report was prepared assuming that Giles will perform Construction Materials Testing (CMT) services during construction of the proposed development. In general, CMT services are recommended (and expected) to at least include observation and testing of foundation and pavement support soil and other construction materials. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

6.8 Basis of Report

This report is based on Giles' proposal, which is dated April 2, 2024 and is referenced by Giles' proposal number 2GP-2403019-R. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

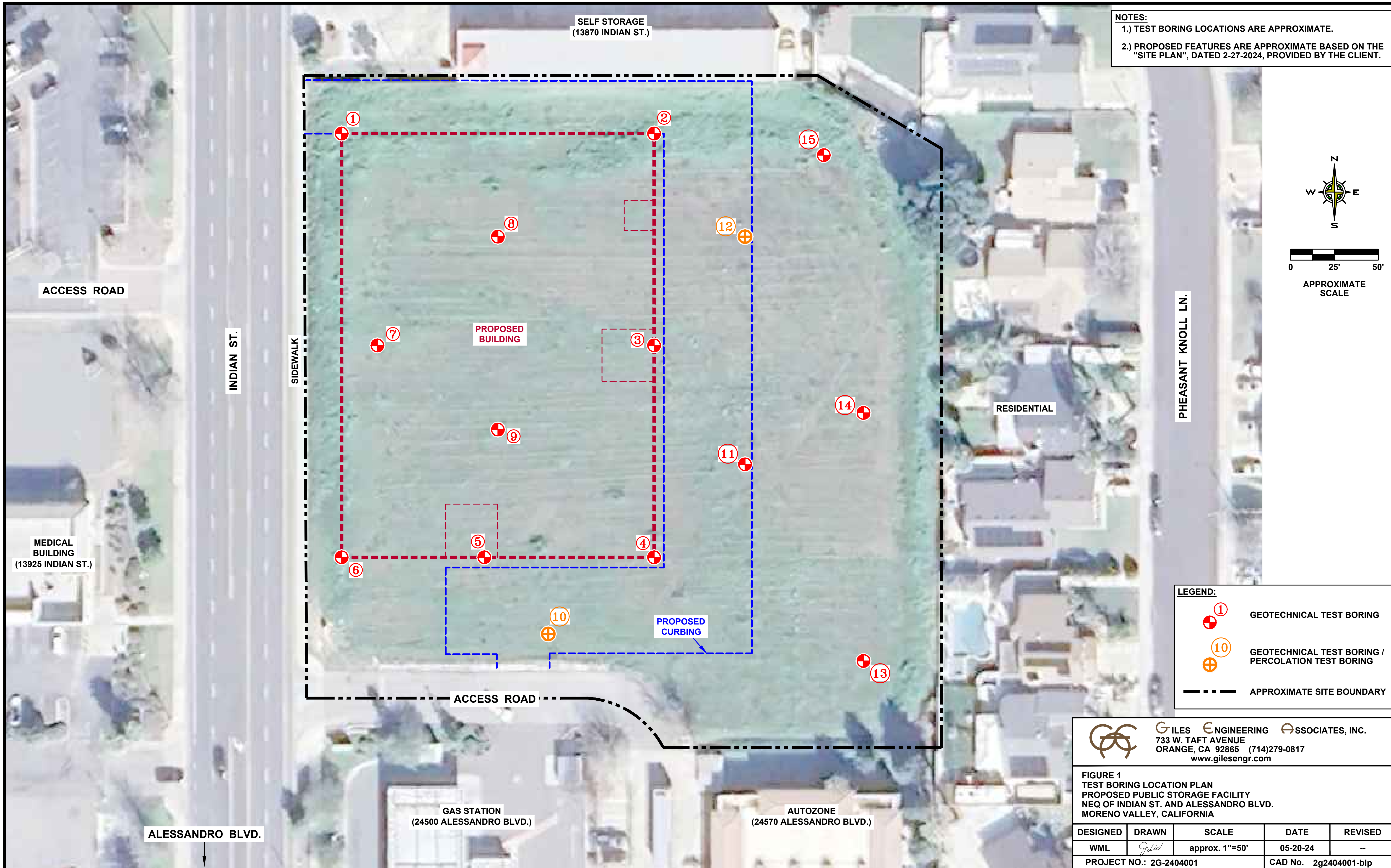
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Records of Subsurface Exploration* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

APPENDIX A

FIGURES AND TEST BORING LOGS

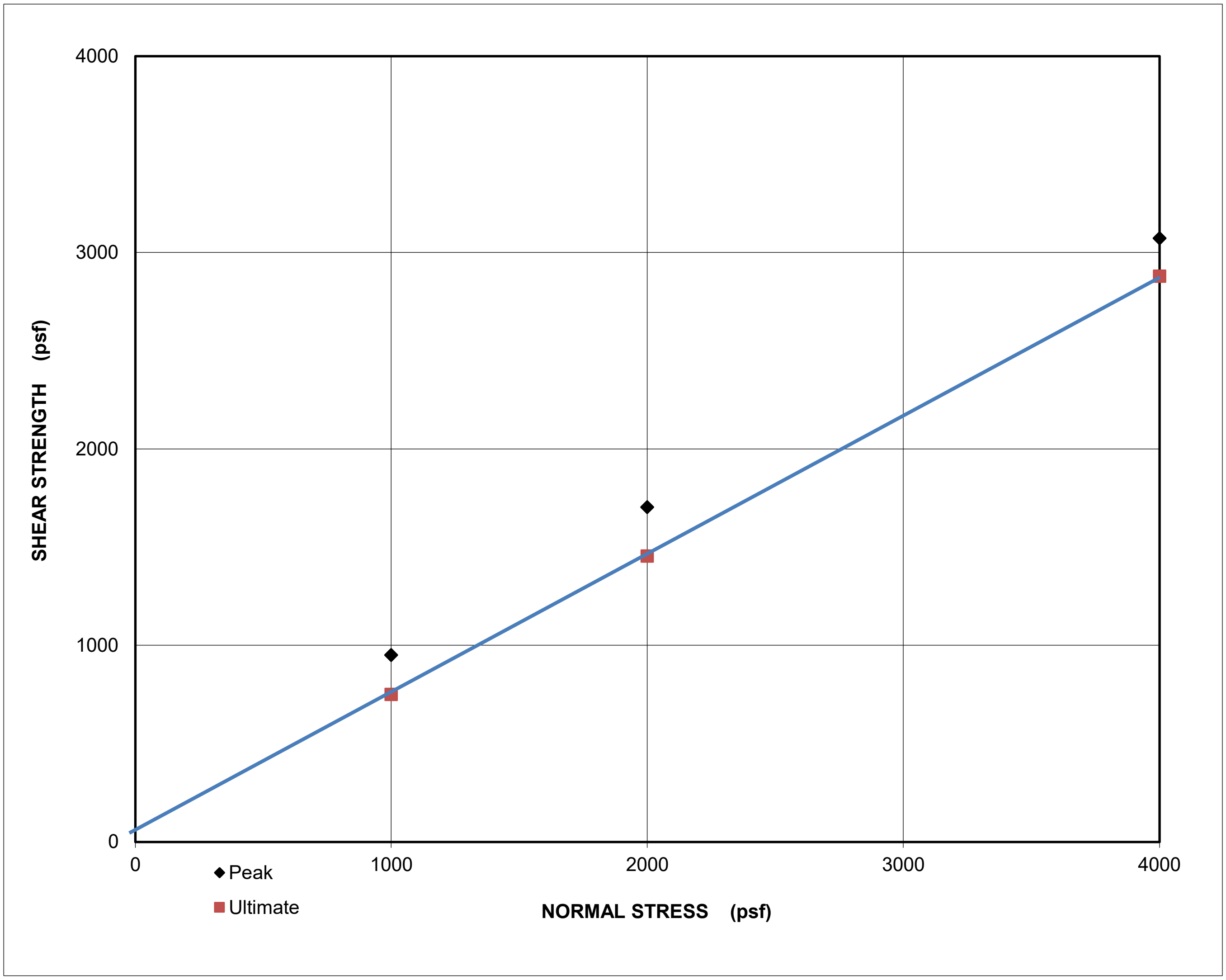
The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.



SELF STORAGE
(13870 INDIAN ST.)
NOTES:
1.) TEST BORING LOCATIONS ARE APPROXIMATE.
2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "SITE PLAN", DATED 2-27-2024, PROVIDED BY THE CLIENT.
N
W
E
S
0
25'
50'
APPROXIMATE SCALE
ACCESS ROAD
INDIAN ST.
SIDEWALK
PROPOSED BUILDING
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
RESIDENTIAL
PHEASANT KNOLL LN.
MEDICAL BUILDING
(13925 INDIAN ST.)
PROPOSED CURBING
ACCESS ROAD
GAS STATION
(24500 ALESSANDRO BLVD.)
AUTOZONE
(24570 ALESSANDRO BLVD.)
ALESSANDRO BLVD.
LEGEND:
GEOTECHNICAL TEST BORING
GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING
APPROXIMATE SITE BOUNDARY
GILES ENGINEERING ASSOCIATES, INC.
733 W. TAFT AVENUE
ORANGE, CA 92865 (714)279-0817
www.gilesengr.com
FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED PUBLIC STORAGE FACILITY
NEQ OF INDIAN ST. AND ALESSANDRO BLVD.
MORENO VALLEY, CALIFORNIA
DESIGNED WML
DRAWN
SCALE approx. 1"=50'
DATE 05-20-24
REVISED --
PROJECT NO.: 2G-2404001
CAD No. 2g2404001-blp

DIRECT SHEAR TEST (ASTM D3080)

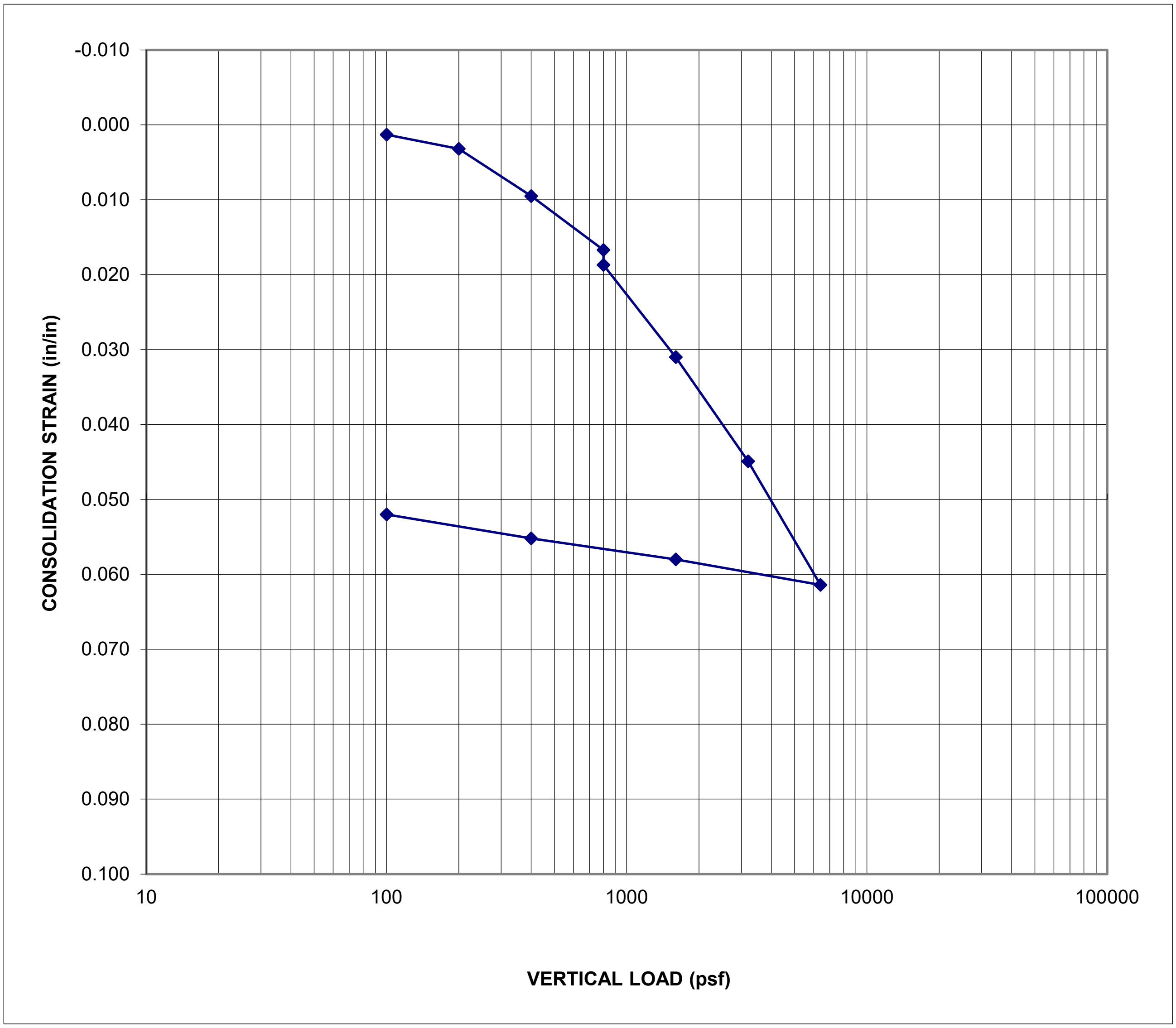


Boring No.	**B-6**	Depth	**3.5 -ft**
Soil Type:	(SP-SM) - Poorly-graded Sand with Silt		
Sample Condition:	Saturated	Initial Specimen Properties:	
		Diameter (in.)	2.4
		Height (in.)	1.0
		In-Situ Moisture Content (%)	4.1
PROJECT:	**Public Storage Facility**	Remolded Density (pcf)	N/A
	Moreno Valley, CA	Dry Density (pcf)	104.3
CLIENT:	**Public Storage**	LL	n/a
PROJECT NO.:	**2G-2404001**	PL	n/a
DATE:	May 2024	C (psf)	40
		PHI (degrees)-Ultimate	35

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

FIGURE No.: 2

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SP-SM) - Poorly-graded Sand with Silt		
Boring No.	**B-6**		
Sample No.	2-CS	Initial Moisture Content (%)	4.1
Depth (ft.)	**3.5 ft**	Final Moisture Content (%)	13.2
Elevation (ft.)	1576	Natural Density (pcf)	111.2
Liquid Limit	N/A	Initial Dry Density (pcf)	106.8
Plastic Limit	N/A	Final Dry Density (pcf)	107.0
Specimen Diameter (in.)	2.42	Collapse at 800 psf	0.20%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project:	**Public Storage Facility** Moreno Valley, CA
Client:	**Public Storage**
Project No.:	**2G-2404001**
Figure No.:	**3**

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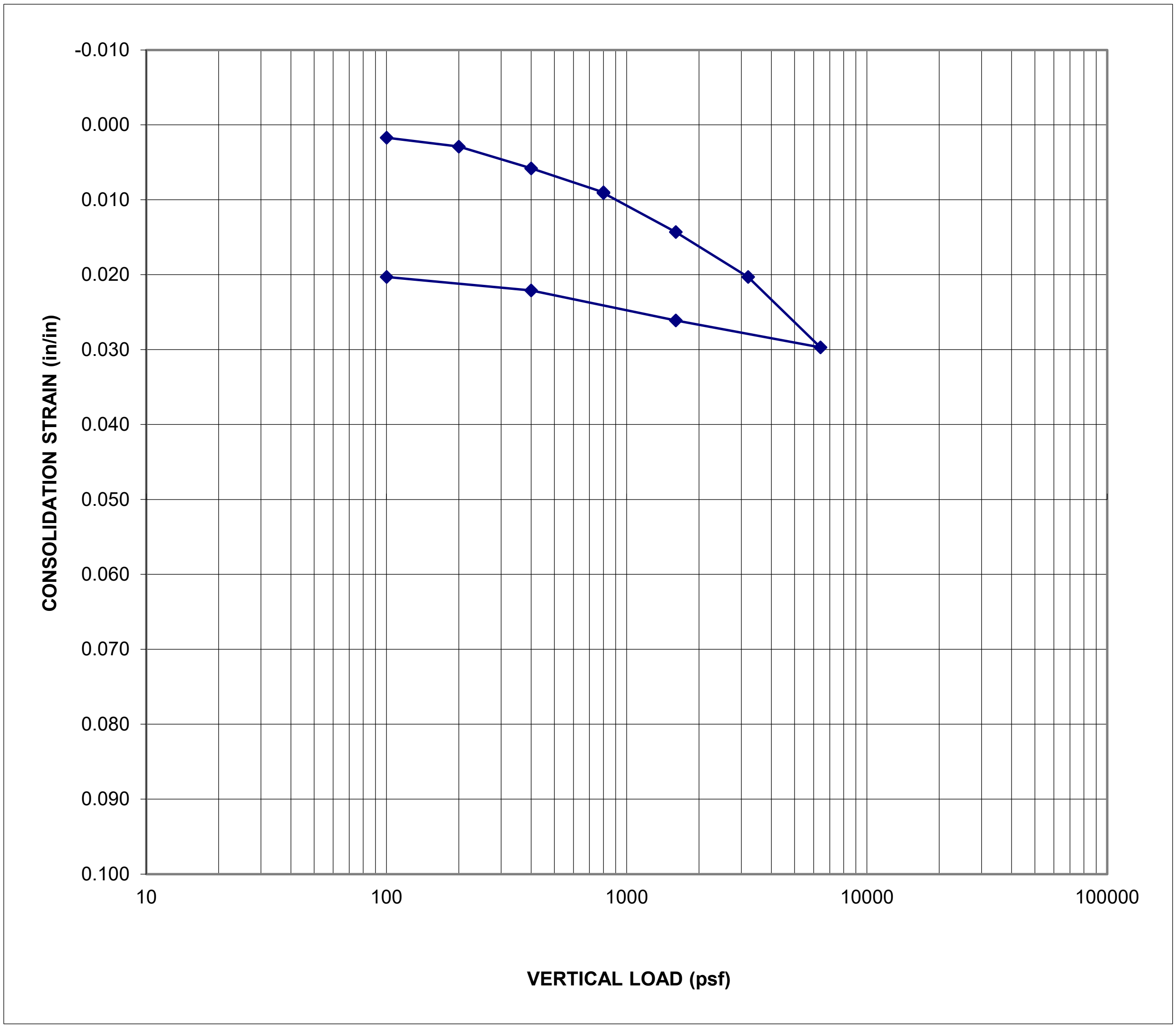
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SM) - Silty Sand		
Boring No.	**B-14**		
Sample No.	2-CS	Initial Moisture Content (%)	9.3
Depth (ft.)	**3.5 ft**	Final Moisture Content (%)	13.5
Elevation (ft.)	1579	Natural Density (pcf)	127.6
Liquid Limit	N/A	Initial Dry Density (pcf)	116.8
Plastic Limit	N/A	Final Dry Density (pcf)	116.8
Specimen Diameter (in.)	2.42	Collapse at 800 psf	0.01%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project: **Public Storage Facility**
Moreno Valley, CA
Client: **Public Storage**

Project No.: **2G-2404001**

Figure No.: **4**

GILES ENGINEERING ASSOCIATES, INC.

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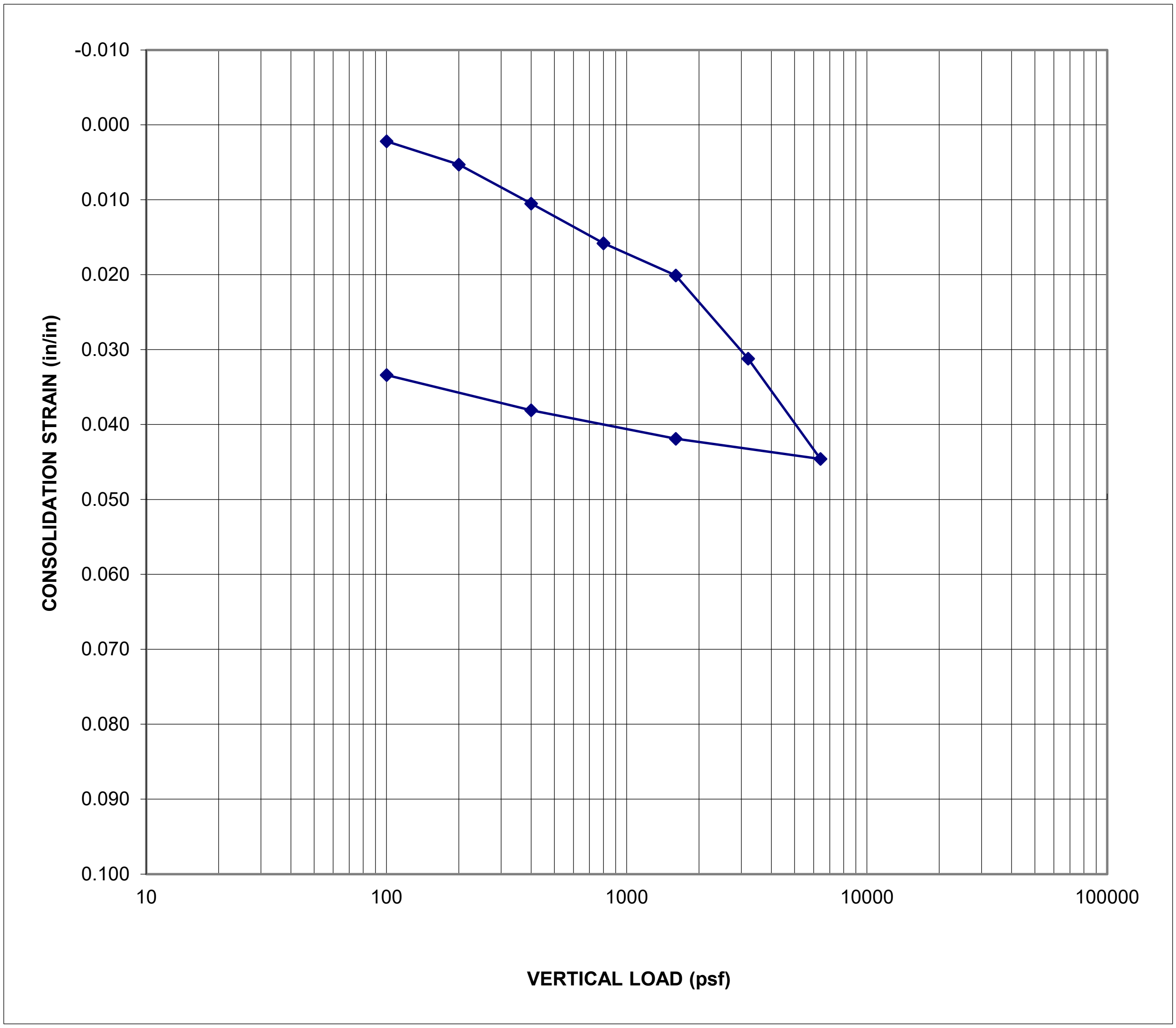
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SM) - Silty Sand		
Boring No.	**B-15**		
Sample No.	3-CS	Initial Moisture Content (%)	3.8
Depth (ft.)	**6 ft**	Final Moisture Content (%)	13.5
Elevation (ft.)	1577	Natural Density (pcf)	115.4
Liquid Limit	N/A	Initial Dry Density (pcf)	111.2
Plastic Limit	N/A	Final Dry Density (pcf)	111.2
Specimen Diameter (in.)	2.42	Collapse at 800 psf	0.00%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project:	**Public Storage Facility** Moreno Valley, CA
Client:	**Public Storage**
Project No.:	**2G-2404001**
Figure No.:	**5**

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
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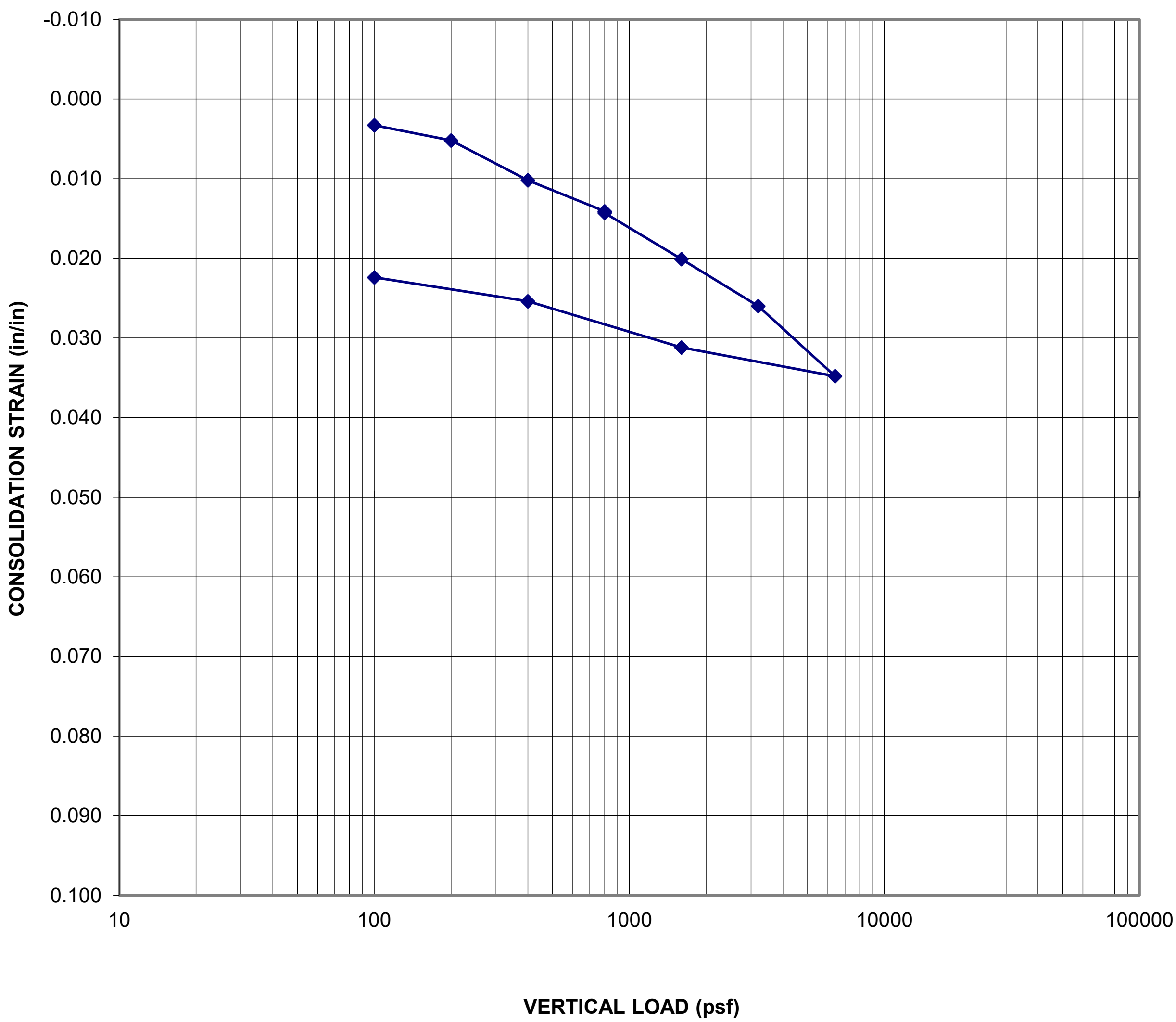
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SC) - Clayey Sand		
Boring No.	**B-1**		
Sample No.	4-CS	Initial Moisture Content (%)	10.0
Depth (ft.)	**10 ft**	Final Moisture Content (%)	12.2
Elevation (ft.)	1573	Natural Density (pcf)	137.1
Liquid Limit	N/A	Initial Dry Density (pcf)	124.6
Plastic Limit	N/A	Final Dry Density (pcf)	124.6
Specimen Diameter (in.)	2.42	Collapse at 800 psf	0.02%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project: **Public Storage Facility**
Moreno Valley, CA

Client: **Public Storage**

Project No.: **2G-2404001**

Figure No.: **6**

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-

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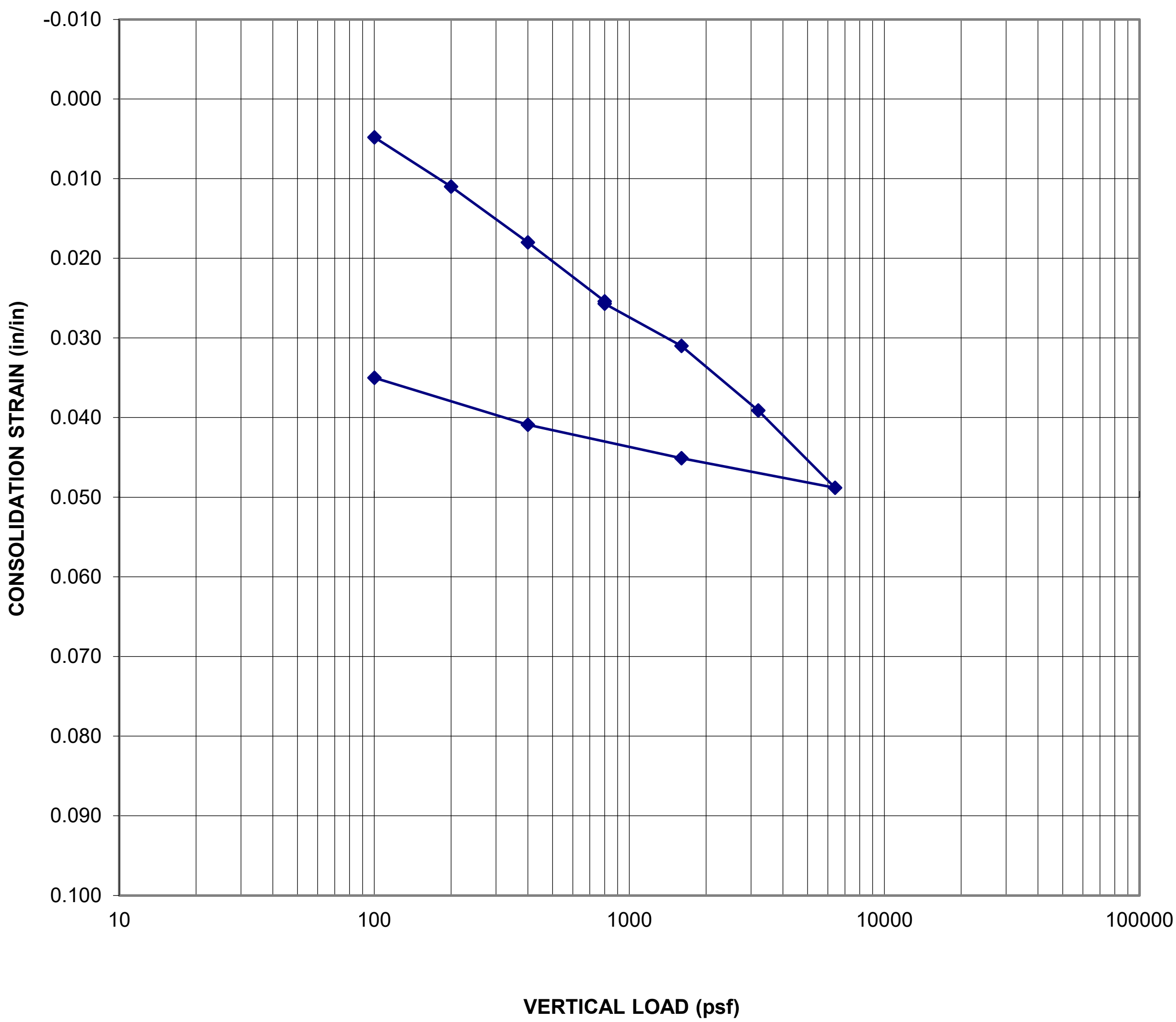
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SM) - Silty Sand		
Boring No.	**B-13**		
Sample No.	4-CS	Initial Moisture Content (%)	2.4
Depth (ft.)	**10 ft**	Final Moisture Content (%)	12.3
Elevation (ft.)	1571	Natural Density (pcf)	118.8
Liquid Limit	N/A	Initial Dry Density (pcf)	116.0
Plastic Limit	N/A	Final Dry Density (pcf)	116.0
Specimen Diameter (in.)	2.42	Collapse at 800 psf	0.03%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project:	**Public Storage Facility** Moreno Valley, CA
Client:	**Public Storage**
Project No.:	**2G-2404001**
Figure No.:	**7**

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
733 W. TAFT AVENUE, ORANGE, CALIFORNIA
OFFICE: 714-279-0817 FAX : 714-279-9687

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
B- 1		
SURFACE ELEVATION: 1583 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, trace fine gravel, trace clay- Moist			1-SS	21				7		
		1580	2-SS	13				7		P_{200}=18%
	5									
Brown, Clayey Sand, fine to medium grained, some coarse- Moist			3-SS	4				6		
		1575								
Fine grained, some medium	10		4-CS	40				10		Dd=121.0 pcf
		1570								
Brown, Sandy Silt, fine Sand, some Clay, some Mica, trace gravel- Moist	15		5-SS	28		4.5		11		
Boring Terminated at about 16.5 feet (EL. 1566.5')										

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test
Water Level At End of Drilling:	CS= California Split Spoon
Cave Depth At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
Water Level After __ Hours: __ ft.	Elevations based on Google Earth
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 2	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1582 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, some coarse, some Mica- Moist		1580	1-SS	9		1.5		8		
Brown, fine grained, some Mica- Moist			2-SS	13				7		
Light Brown, Sand, fine to medium grained, some coarse grain- Moist	5	1575	3-SS	7				3		P_{200}=3%
Light Brown, Silty Sand, fine to medium grained, some coarse, some fine gravel- Moist	10	1570	4-SS	11				3		
Dark Brown, Sandy Silt, fine Sand, trace medium Sand, trace Clay, some Mica- Moist	15		5-SS	26		3.75		12		

Boring Terminated at about 16.5 feet (EL. 1565.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
▬	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 3	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1582 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Sandy Silt, fine Sand, some medium, trace Clay, some Mica- Moist		1580	1-SS	2				15		
Brown, Silty Sand, fine to medium grained, trace Clay- Moist	5		2-SS	4				14		
Orange, Browm, fine to medium grained, some coarse		1575	3-SS	4				11		
Trace Gravel	10	1570	4-SS	7				12		
Yellowish Brown, fine to medium grained, some Mica	15		5-SS	13				16		

Boring Terminated at about 16.5 feet (EL. 1565.5')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
▬	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 4	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1580 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Orange, Brown, Silty Sand, fine to medium grained, trace Clay- Moist			1-SS	3				16		
Light Brown, poorly-graded Sand, fine to medium grained, some coarse, trace Gravel- Moist	5	1575	2-SS	7				11		P_{200}=5%
Brownish Gray, fine to coarse grained, some Silt, some fine Gravel- Moist			3-CS	17				5		Dd=103.4 pcf
Brown, Sandy Silt, fine Sand, some Mica- Moist	10	1570	4-SS	18		1.5		19		
Some Clay- Moist	15	1565	5-SS	25		2.25		21		

Boring Terminated at about 16.5 feet (EL. 1563.5')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: None	SS= Standard Penetration Test CS= California Split Spoon Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
▬	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 5	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1579 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, trace fine Gravel and Clay- Moist			1-SS	4				8		
Light Brown	5	1575	2-SS	5				6		
Orange Brown, some Clay, fine to medium grained		1570	3-SS	7				7		
Yellowish Brown, poorly graded sand with Silt, some coarse grained, trace fine Gravel- slightly Moist	10	1565	4-SS	19				3		
Brown, Clayey Silt, some fine Sand, some Mica- Moist	15	1560	5-SS	17		4.0		13		
Yellowish Brown, Sandy Clay, some Silt, fine Sand, some Mica- Moist	20		6-SS	17		2.5		11		
Boring Terminated at about 20 feet (EL. 1559')										

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test
Water Level At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
Cave Depth At End of Drilling:	Elevations based on Google Earth
Water Level After __ Hours: __ ft.	
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 6	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1580 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, some coarse, trace Clay- Moist			1-SS	7				7		
Light Brown			2-CS	8				4		Dd=104.3 pcf
	5	1575								
Orange Brown, fine to medium grained, some coarse grained, trace fine Gravel			3-SS	8				6		
	10	1570								
Light Brown, trace fine Gravel- slightly Moist			4-SS	9				4		
	15	1565								
Brown, Sandy Silt, fine Sand, some Clay, some Mica- Moist			5-SS	25		2.0		13		
Boring Terminated at about 16.5 feet (EL. 1563.5')										

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS= Standard Penetration Test CS= California Split Spoon Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
▓	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 7	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1581 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, some coarse, trace fine Gravel, some Mica- Moist		1580	1-SS	2				9		
Light Brown	5		2-SS	4				5		
Light Brown, Sandy Silt, fine Sand, trace fine Gravel, some Mica- Moist		1575	3-SS	9		1.0		7		
Some Clay	10	1570	4-SS	25		2.5		9		
Brown, Sandy Clay, some Silt, fine Sand, some Mica- Moist	15	1565	5-SS	28		4.5		13		
Boring Terminated at about 16.5 feet (EL. 1564.5')										

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test
Water Level At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
Cave Depth At End of Drilling:	Elevations based on Google Earth
Water Level After __ Hours: __ ft.	
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	
B- 8		
SURFACE ELEVATION: 1583 feet	PUBLIC STORAGE FACILITY	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, some coarse, some Mica, trace Clay- very Moist			1-SS	8				9		
		1580								
Orange Brown			2-SS	6				7		
	5									
More coarse grained, trace fine Gravel			3-SS	14				5		
		1575								
	10		4-SS	12				5		
		1570								
Brown, Sandy Silt, fine Sand, trace fine Gravel, some Mica- Moist	15		5-SS	58				9		
		1565								
Yellowish Brown, Sandy Clay, fine Sand, some Mica- Moist	20		6-SS	20		4.5		13		
		1560								
Yellowish Brown, Silty Sand, fine to medium grained, trace coarse- slightly Moist	25		7-SS	18				4		
		1555								
Light Brown, Sandy Silt, fine Sand, some Mica, weakly cemented- Moist	30		8-SS	22				10		
		1550								
Olive Brown, Lean Clay, trace fine Sand, trace Silt, moderately plastic- very Moist	35 ▽		9-SS	31		1.0		23		
		1545								
	40									

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 36 ft.	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	
B- 8		
SURFACE ELEVATION: 1583 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	**GILES ENGINEERING ASSOCIATES, INC.**
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light Brown, Clayey Sand, fine to medium grained, trace coarse- very Moist		1540	10-SS	23				12		P_{200}=16%
Brown, Lean Clay, trace fine Sand, trace Silt, moderately plastic- Very Moist	45	1535	11-SS	22		4.0		19		P_{200}=51%
Brown, Sandy Clay, fine Sand, some medium grained- very Moist	50		12-SS	32		3.25		14		
Boring Terminated at about 50 feet (EL. 1533')										

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: 36 ft.	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After __ Hours: __ ft.	
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B- 9	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1580 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Orange Brown, Silty Sand, fine to medium grained- Moist			1-SS	4				7		
Light Brown, fine to medium grained, trace coarse, trace fine Gravel	5	1575	2-SS	6				6		
More coarse grained			3-SS	13				4		
Brownish Gray, poorly graded Sand withSilt, fine to medium grained, some coarse, trace fine Gravel- slightly Moist	10	1570	4-SS	22				4		
Brown, Sandy Clay, fine Sand, trace Calcium Carbonate- Moist	15	1565	5-SS	21		>4.5		12		

Boring Terminated at about 16.5 feet (EL. 1563.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	Water Level After __ Hours: __ ft.	
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: B-10	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1579 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Silty Sand, fine to medium grained, trace coarse, trace Clay- Moist			1-SS	3				7		
More fine Gravel	5	1575	2-SS	6				6		
Light Brown, fine to medium grained, some coarse grained, trace fine Gravel			3-SS	10				5		
Light Brown, poorly graded Sand, some Silt, fine to medium grained, some coarse- slightly Moist	10	1570	4-SS	16						P_{200}=4%

Boring Terminated at about 10 feet (EL. 1569')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After __ Hours: __ ft.	
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

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BORING NO. & LOCATION: B-11	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 1582 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Sandy Silt, fine Sand, trace medium Sand, some Mica, weakly cemented, fine Gravel- Moist (Possible Fill)		1580	1-SS	13		3.0		10		
Brown, Sandy Silt, fine Sand, trace fine Gravel, trace Clay- Moist (Native)	5		2-SS	3				9		
		1575	3-SS	4				8		
Light Brown, Silty Sand, fine to medium grained, trace coarse- Moist	10		4-SS	6				5		

Boring Terminated at about 11.5 feet (EL. 1570.5')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After __ Hours: __ ft.	
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
B-12	PUBLIC STORAGE FACILITY	
SURFACE ELEVATION: 1582 feet	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
COMPLETION DATE: 04/17/24		
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown, Sandy Silt, fine Sand, some medium- Moist		1580	1-SS	7				10		
	5		2-SS	2		1.25		10		
Brown, Silty Sand, fine grained, trace medium, some Mica- Moist		1575	3-SS	5				7		
Yellow, Brown, poorly graded Sand, some Silt, fine to medium grained, some coarse- Moist	10		4-SS	4				3		P_{200}=3%
Boring Terminated at about 10 feet (EL. 1572')										

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test
Water Level At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
Cave Depth At End of Drilling:	Elevations based on Google Earth
Water Level After __ Hours: __ ft.	
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	
B-13		
SURFACE ELEVATION: 1581 feet	PUBLIC STORAGE FACILITY	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brown, Sandy Silt, fine to medium Sand, trace fine Gravel- Moist (Possible Fill)		1580	1-SS	41		2.5		7		
Brown, Sandy Silt, some Clay, fine Sand, some Mica- Moist (Native)	5		2-SS	2		1.25		10		
Light Brown, Silty Sand, fine to medium grained, some coarse (No Recovery)		1575								
Yellow, Brown, fine to medium grained, some coarse, some fine Gravel- slightly Moist	10	1570	4-SS	21				2		Dd=112.5 pcf
	15	1565	5-SS	18						

Boring Terminated at about 16.5 feet (EL. 1564.5')

	Water Observation Data	Remarks:
	Water Encountered During Drilling: None	SS= Standard Penetration Test Drilling Equipment used: Hollow stem auger; 8-in diameter Elevations based on Google Earth
	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
	Water Level After __ Hours: __ ft.	
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
B-14	PUBLIC STORAGE FACILITY	
SURFACE ELEVATION: 1583 feet	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
COMPLETION DATE: 04/17/24		
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown, Silty Sand, fine to medium grained, some coarse, trace Clay, some Mica- Moist (Possible Fill)			1-SS	15				3		
		1580								
Brown, Silty Sand, fine to medium grained, some coarse, some Clay- Moist	5		2-CS	8				9		
No Recovery			3-SS	N/A						
		1575								
Light Brown- slightly Moist	10		4-SS	9				3		
		1570								
Brown, Sandy Silt, fine Sand, some Mica, trace Clay- Moist	15		5-SS	37		2.0		15		
Boring Terminated at about 16.5 feet (EL. 1566.5')										

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS= Standard Penetration Test
Water Level At End of Drilling:	CS= California Split Spoon
Cave Depth At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
Water Level After __ Hours: __ ft.	Elevations based on Google Earth
Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	TEST BORING LOG	
B-15		
SURFACE ELEVATION: 1583 feet	PUBLIC STORAGE FACILITY	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 04/17/24	NEQ OF INDIAN STREET & ALESSANDRO BOULEVARD MORENO VALLEY, CALIFORNIA	
FIELD REP: DOUGLAS CALLEJAS	PROJECT NO: 2G-2404001	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Dark Brown, Sandy Silt, fine Sand, trace fine Gravel, some mica- Moist (Possible Fill)			1-SS	6		1.0		10		
		1580								
Brown, Sandy Silt, fine Sand -Moist (Native)			2-SS	4				5		
	5									
Light Brown, Silty Sand, fine grained, trace Clay, fine Gravel- Moist			3-CS	11				4		Dd=115.6 pcf
		1575								
	10									
Yellow Brown, fine to medium grained, some coarse- slightly Moist			4-SS	7				3		
		1570								
	15									
Brown, Sandy Clay, fine Sand, weakly cemented- Moist			5-SS	31		2.75		9		

Boring Terminated at about 16.5 feet (EL. 1566.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS= Standard Penetration Test
▼	Water Level At End of Drilling:	CS= California Split Spoon
	Cave Depth At End of Drilling:	Drilling Equipment used: Hollow stem auger; 8-in diameter
▼	Water Level After __ Hours: __ ft.	Elevations based on Google Earth
	Cave Depth After __ Hours: __ ft.	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

GILES LOG REPORT 2G-2404001.GPJ GILES.GDT 5/23/24

Project name (number):	2G-2404001_PS - Moreno Valley
Engineer:	WL
Reviewer:	
Date:	05/22/24

Soil boring number:	**B-8**	(A)
Approximate borehole diameter (inch):	8.0	(B)
Soil boring ground surface elevation (ft):	100.0	(C)
Groundwater depth during field exploration (ft):	36.0	(D)
Design groundwater depth (ft):	35.0	(E)
Design fill/cut, H (ft): {Positive for fill, negative for cut}	0.0	(F)
Unit weight of fill/cut material (pcf):	120	(G)
SPT hammer energy ratio (ER): {Enter percentage}	81.5	(H)
Liner used in sampler? {Yes or No}	No	(I)
Plasticity index cutoff value for liquefaction (%)	12	(J)
Peak horizontal ground acceleration (a_{max}/g):	0.737	(K)
Design earthquake magnitude (M_w):	7.38	(L)
Factor of safety criterion: {1.0 or higher}	1.3	(M)
Risk Level	2% in 50 yrs	

Total Settlement
0.0

Hammer Energy Ratios		
Hammer Type	Range [1]	Recommend [2]
Safety	42% - 72%	60%
Donut	30% - 60%	50%
Automatic-Trip	48% - 78%	80%

1) after recommendations by NCEER, 1997
2) for good-quality equipment and procedures conforming to ASTM D-1686.

PLATE A-1

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	
																										Tokimatsu	Ishihara & Yoshimine (1992)		Tokimatsu & Seed (1987)		
Interval Bottom Depth (ft)	Interval Soil Type	Total Unit Weight (pcf)	Field SPT N Value N_m	Estimated Fines Content FC (%)	Estimated Plasticity Index (%)	Relative Density Dr (%)	Restricted Relative Density D_r (%)	Interval Bottom Elevation (ft)	Interval Thickness (ft)	Effective Unit Weight (pcf)	Field Total Overburden (tsf)	Design Total Overburden (tsf)	Field Effective Overburden (tsf)	Design Effective Overburden (tsf)	Effective Overburden Correction C_N	Hammer Energy Correction C_E	Borehole Diameter Correction C_B	Rod Length Correction C_R	Sampling Method Correction C_S	Energy Correction Only N_{60}	No Fines Correction $(N_1)_{60}$	Clean Sand $(N_1)_{60cs}$	Factor of Safety FS	Liquefaction Susceptibility	Comments	Cyclic Stress Ratio $CSR_{7.5}$	Volumetric Strain (%)	Seismic Settlement (inches)	Volumetric Strain (%)	Seismic Settlement (inches)	Elevation Feet
0											0	0.000	0	0.000																	
0.5		120	8			59	59	99.5	0.5	57.6	0.03	0.03	0.03	0.03	1.7	1.36	1.15	0.75	1.15	11	18	18	0.47	NO	Above GWT	0.46	0.000	0.000	0.000	0.000	99.5
1		120	8			59	59	99.0	0.5	57.6	0.060	0.060	0.060	0.060	1.70	1.36	1.15	0.75	1.15	11	18	18	0.47	NO	Above GWT	0.459	0.000	0.000	0.000	0.000	99.0
1.5		120	8			59	59	98.5	0.5	57.6	0.090	0.090	0.090	0.090	1.70	1.36	1.15	0.75	1.15	11	18	18	0.47	NO	Above GWT	0.458	0.000	0.000	0.000	0.000	98.5
2		120	8			59	59	98.0	0.5	57.6	0.120	0.120	0.120	0.120	1.68	1.36	1.15	0.75	1.15	11	18	18	0.47	NO	Above GWT	0.458	0.000	0.000	0.000	0.000	98.0
2.5		120	8			58	58	97.5	0.5	57.6	0.15	0.15	0.15	0.15	1.64	1.36	1.15	0.75	1.15	11	18	18	0.46	NO	Above GWT	0.46	0.000	0.000	0.000	0.000	97.5
3		120	8			58	58	97.0	0.5	57.6	0.180	0.180	0.180	0.180	1.61	1.36	1.15	0.75	1.15	11	17	17	0.44	NO	Above GWT	0.457	0.000	0.000	0.000	0.000	97.0
3.5		120	8			57	57	96.5	0.5	57.6	0.210	0.210	0.210	0.210	1.57	1.36	1.15	0.75	1.15	11	17	17	0.43	NO	Above GWT	0.456	0.000	0.000	0.000	0.000	96.5
4		120	6	13		53	53	96.0	0.5	57.6	0.240	0.240	0.240	0.240	1.54	1.36	1.15	0.75	1.15	8	12	15	0.38	NO	Above GWT	0.456	0.000	0.000	0.000	0.000	96.0
4.5		120	6	13		53	53	95.5	0.5	57.6	0.270	0.270	0.270	0.270	1.51	1.36	1.15	0.75	1.15	8	12	15	0.37	NO	Above GWT	0.455	0.000	0.000	0.000	0.000	95.5
5		120	6	13		52	52	95.0	0.5	57.6	0.300	0.300	0.300	0.300	1.48	1.36	1.15	0.75	1.15	8	12	14	0.37	NO	Above GWT	0.454	0.000	0.000	0.000	0.000	95.0
5.5		120	6	13		52	52	94.5	0.5	57.6	0.330	0.330	0.330	0.330	1.46	1.36	1.15	0.75	1.15	8	12	14	0.36	NO	Above GWT	0.454	0.000	0.000	0.000	0.000	94.5
6		120	6	13		52	52	94.0	0.5	57.6	0.360	0.360	0.360	0.360	1.43	1.36	1.15	0.75	1.15	8	12	14	0.36	NO	Above GWT	0.453	0.000	0.000	0.000	0.000	94.0
6.5		120	14	13		75	75	93.5	0.5	57.6	0.390	0.390	0.390	0.390	1.40	1.36	1.15	0.75	1.15	19	26	29	3.71	NO	Large factor of safety	0.453	0.000	0.000	0.000	0.000	93.5
7		120	14			71	71	93.0	0.5	57.6	0.420	0.420	0.420	0.420	1.38	1.36	1.15	0.75	1.15	19	26	26	0.97	NO	Above GWT	0.452	0.000	0.000	0.000	0.000	93.0
7.5		120	14			70	70	92.5	0.5	57.6	0.450	0.450	0.450	0.450	1.35	1.36	1.15	0.75	1.15	19	26	26	0.90	NO	Above GWT	0.452	0.000	0.000	0.000	0.000	92.5
8		120	14			69	69	92.0	0.5	57.6	0.480	0.480	0.480	0.480	1.33	1.36	1.15	0.75	1.15	19	25	25	0.85	NO	Above GWT	0.451	0.000	0.000	0.000	0.000	92.0
8.5		120	14			69	69	91.5	0.5	57.6	0.510	0.510	0.510	0.510	1.31	1.36	1.15	0.75	1.15	19	25	25	0.81	NO	Above GWT	0.451	0.000	0.000	0.000	0.000	91.5
9		120	14			68	68	91.0	0.5	57.6	0.540	0.540	0.540	0.540	1.29	1.36	1.15	0.75	1.15	19	24	24	0.78	NO	Above GWT	0.450	0.000	0.000	0.000	0.000	91.0
9.5		120	14			68	68	90.5	0.5	57.6	0.570	0.570	0.570	0.570	1.27	1.36	1.15	0.75	1.15	19	24	24	0.75	NO	Above GWT	0.450	0.000	0.000	0.000	0.000	90.5
10		120	14			69	69	90.0	0.5	57.6	0.600	0.600	0.600	0.600	1.25	1.36	1.15	0.80	1.15	19	25	25	0.85	NO	Above GWT	0.449	0.000	0.000	0.000	0.000	90.0
10.5		120	12	7		64	64	89.5	0.5	57.6	0.630	0.630	0.630	0.630	1.23	1.36	1.15	0.80	1.15	16	21	21	0.60	NO	Above GWT	0.449	0.000	0.000	0.000	0.000	89.5
11		120	12	7		64	64	89.0	0.5	57.6	0.660	0.660	0.660	0.660	1.21	1.36	1.15	0.80	1.15	16	21	21	0.59	NO	Above GWT	0.448	0.000	0.000	0.000	0.000	89.0
11.5		120	12	7		63	63	88.5	0.5	57.6	0.690	0.690	0.690	0.690	1.19	1.36	1.15	0.80	1.15	16	20	21	0.58	NO	Above GWT	0.448	0.000	0.000	0.000	0.000	88.5
12		120	12	7		63	63	88.0	0.5	57.6	0.720	0.720	0.720	0.720	1.17	1.36	1.15	0.80	1.15	16	20	20	0.56	NO	Above GWT	0.447	0.000	0.000	0.000	0.000	88.0
12.5		120	12	7		62	62	87.5	0.5	57.6	0.750	0.750	0.750	0.750	1.15	1.36	1.15	0.80	1.15	16	20	20	0.55	NO	Above GWT	0.446	0.000	0.000	0.000	0.000	87.5
13		120	12	7		62	62	87.0	0.5	57.6	0.780	0.780	0.780	0.780	1.14	1.36	1.15	0.80	1.15	16	20	20	0.54	NO	Above GWT	0.446	0.000	0.000	0.000	0.000	87.0
13.5		120	12	7		63	63	86.5	0.5	57.6	0.810	0.810	0.810	0.810	1.12	1.36	1.15	0.85	1.15	16	21	21	0.58	NO	Above GWT	0.445	0.000	0.000	0.000	0.000	86.5
14		120	12	7		63	63	86.0	0.5	57.6	0.840	0.840	0.840	0.840	1.10	1.36	1.15	0.85	1.15	16	20	21	0.57	NO	Above GWT	0.445	0.000	0.000	0.000	0.000	86.0
14.5		120	12	7		62	62	85.5	0.5	57.6	0.870	0.870	0.870	0.870	1.09	1.36	1.15	0.85	1.15	16	20	20	0.56	NO	Above GWT	0.444	0.000	0.000	0.000	0.000	85.5
15		120	12	7		62	62	85.0	0.5	57.6	0.900	0.900	0.900	0.900	1.07	1.36	1.15	0.85	1.15	16	20	20	0.55	NO	Above GWT	0.444	0.000	0.000	0.000	0.000	85.0
15.5		120	58			134	100	84.5	0.5	57.6	0.930	0.930	0.930	0.930	1.06	1.36	1.15	0.85	1.15	79	94	94	>4	NO	Large factor of safety	0.443	0.000	0.000	0.000	0.000	84.5
16		120	58			133	100	84.0	0.5	57.6	0.960	0.960	0.960	0.960	1.04	1.36	1.15	0.85	1.15	79	92	92	>4	NO	Large factor of safety	0.443	0.000	0.000	0.000	0.000	84.0
16.5		120	58			132	100	83.5	0.5	57.6	0.990	0.990	0.990	0.990	1.03	1.36	1.15	0.85	1.15	79	91	91	>4	NO	Large factor of safety	0.442	0.000	0.000	0.000	0.000	83.5
17		120	58			132	100	83.0	0.5	57.6	1.020	1.020	1.020	1.020	1.02	1.36	1.15	0.85	1.15	79	90	90	>4	NO	Large factor of safety	0.442	0.000	0.000	0.000	0.000	83.0
17.5		120	58			131	100	82.5	0.5	57.6	1.050	1.050	1.050	1.050	1.00	1.36	1.15	0.85	1.15	79	89	89	>4	NO	Large factor of safety	0.441	0.000	0.000	0.000	0.000	82.5
18		120	58			130	100	82.0	0.5	57.6	1.080	1.080	1.080	1.080	0.99	1.36	1.15	0.85	1.15	79	88	88	>4	NO	Large factor of safety	0.441	0.000	0.000	0.000	0.000	82.0
18.5		120	58			129	100	81.5	0.5	57.6	1.110	1.110	1.110	1.110	0.98	1.36	1.15	0.85	1.15	79	87	87	>4	NO	Large factor of safety	0.440	0.000	0.000	0.000	0.000	81.5
19		120	58			128	100	81.0	0.5	57.6	1.140	1.140	1.140	1.140	0.97	1.36	1.15	0.85	1.15	79	86	86	>4	NO	Large factor of safety	0.439	0.000	0.000	0.000	0.000	81.0
19.5		120	58			127	100	80.5	0.5	57.6	1.170	1.170	1.170	1.170	0.95	1.36	1.15	0.85	1.15	79	85	85	>4	NO	Large factor of safety	0.439	0.000	0.000	0.000	0.000	80.5
20		120	58			134	100	80.0	0.5	57.6	1.200	1.200	1.200	1.200	0.94	1.36	1.15	0.95	1.15	79	93	93	>4	NO	Large factor of safety	0.438	0.000	0.000	0.000	0.000	80.0
20.5		120	20			78	78	79.5	0.5	57.6	1.230	1.230	1.230	1.230	0.93	1.36	1.15	0.95	1.15	27	32	32	>4	NO	Large factor of safety	0.438	0.000	0.000	0.000	0.000	79.5
21		120	20			78	78	79.0	0.5	57.6	1.260	1.260	1.260	1.260	0.92	1.36	1.15	0.95	1.15	27	31	31	>4	NO	Large factor of safety	0.437	0.000	0.000	0.000	0.000	79.0
21.5		120	20			77	77	78.5	0.5	57.6	1.290	1.290	1.290	1.290	0.91	1.36	1.15	0.95	1.15	27	31	31	>4	NO	Large factor of safety	0.437	0.000	0.000	0.000	0.000	78.5
22		120	20			77	77	78.0	0.5	57.6	1.320	1.320	1.320	1.320	0.90	1.36	1.15	0.95	1.15	27	31	31	>4	NO	Large factor of safety	0.436	0.000	0.000	0.000	0.000	78.0
22.5		120	20			76	76	77.5	0.5	57.6	1.350	1.350	1.350	1.350	0.89	1.36	1.15	0.95	1.15	27	30	30	>4	NO	Large factor of safety	0.436	0.000	0.000	0.000	0.000	77.5
23		120	20			76	76	77.0	0.5	57.6	1.380	1.380	1.380	1.380	0.88	1.36	1.15	0.95	1.15	27	30	30	>4	NO	Large factor of safety	0.435	0.000	0.000	0.000	0.000	77.0
23.5		120	20			76	76	76.5	0.5	57.6	1.410	1.410	1.410	1.410	0.87	1.36	1.15	0.95	1.15	27	30	30	>4	NO	Large factor of safety	0.435	0.000	0.000	0.000	0.000	76.5
24		120	20			75	75	76.0	0.5	57.6	1.440	1.440	1.440	1.440	0.86	1.36	1.15	0.95	1.15	27	29	29	3.88	NO	Large factor of safety	0.434	0.000	0.000	0.000	0.000	76.0
24.5		120	20			75	75	75.5	0.5	57.6	1.470	1.470	1.470	1.470	0.85	1.36	1.15	0.95	1.15	27	29	29	2.79	NO	Large factor of safety	0.434	0.000	0.000	0.000	0.000	75.5
25		120	20			74	74	75.0	0.5	57.6	1.500	1.500	1.500	1.500	0.84	1.36	1.15	0.95	1.15	27	29	29	2.24	NO	Large factor of safety	0.433	0.000	0.000	0.000	0.000	75.0
25.5		120	18			70	70	74.5	0.5	57.6	1.530	1.530	1.530	1.530	0.83	1.36	1.15	0.95	1.15	24	26	26	0.95	NO	Above GWT	0.433	0.000	0.000	0.000	0.000	74.5
26		120	18			70	70	74.0	0.5	57.6	1.560	1.560	1.560	1.560	0.82	1.36	1.15	0.95	1.15	24	25	25	0.91	NO	Above GWT	0.432	0.000	0.000	0.000	0.000	74.0
26.5		120	18			69	69	73.5	0.5	57.6	1.590	1.590	1.590	1.590	0.81	1.36	1.15	0.95	1.15	24	25	25	0.88	NO	Above GWT	0.431	0.000	0.000	0.000	0.000	73.5
27		120	18			69	69	73.0	0.5	57.6	1.620	1.620	1.620	1.620	0.81	1.36	1.15	0.95	1.15	24	25	25	0.86	NO	Above GWT	0.431	0.000	0.000	0.000	0.000	73.0
27.5		120	18			69	69	72.5	0.5	57.6	1.650	1.650	1.650	1.650	0.80	1.36	1.15	0.95	1.15	24	24	24	0.83	NO	Above GWT	0.430	0.000	0.000	0.000	0.000	72.5
28		120	18			68	68	72.0	0.5	57.6	1.680	1.680	1.680	1.680	0.79	1.36	1.15	0.95	1.15	24	24	24	0.81	NO	Above GWT	0.430	0.000	0.000	0.000	0.000	72.0
28.5		120	18			68	68	71.5	0.5	57.6	1.710	1.710	1.710	1.710	0.78	1.36	1.15	0.95	1.15	24	24	24	0.79	NO	Above GWT	0.429	0.000	0.000	0.000	0.000	71.5
29		120	18			68	68	71.0	0.5	57.6	1.740	1.740	1.740	1.740	0.77	1.36	1.15	0.95	1.15	24	24	24	0.77	NO	Above GWT	0.429	0.000	0.000	0.000	0.000	71.0
29.5		120	18			67	67	70.5	0.5	57.6	1.770	1.770	1.770	1.770	0.77	1.36	1.15	0.95	1.15	24	24	24	0.76	NO	Above GWT	0.428	0.000	0.000	0.000	0.000	70.5
30		120	18			67	67	70.0	0.5	57.6	1.800	1.800	1.800	1.800	0.76	1.36	1.15	0.95	1.15	24	23	23	0.74	NO	Above GWT	0.428	0.000	0.000	0.000	0.000	70.0
30.5		120	22			74	74	69.5	0.5	57.6	1.830	1.830	1.830	1.830	0.75	1.36	1.15	0.95	1.15	30	28	28	1.78	NO	Large factor of safety	0.426	0.000	0.000	0.000	0.000	69.5
31		120	22			73	73	69.0	0.5	57.6	1.860	1.860	1.860	1.860	0.74	1.36	1.15	0.95	1.15	30	28	28	1.61	NO	Large factor of safety	0.424	0.000	0.000	0.000	0.000	69.0
31.5		120	22			73	73	68.5	0.5	57.6	1.890	1.890	1.890	1.890	0.74	1.36	1.15	0.95	1.15	30	28	28	1.49	NO	Large factor of safety	0.422	0.000	0.000	0.000	0.000	68.5
32		120	22			73	73	68.0	0.5	57.6	1.920	1.920	1.920	1.920	0.73	1.36	1.15	0.95	1.15	30	27	27	1.39	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	68.0
32.5		120	22			72	72	67.5	0.5	57.6	1.950	1.950	1.950	1.950	0.72	1.36	1.15	0.95	1.15	30	27	27	1.31	NO	Large factor of safety	0.418	0.000	0.000	0.000	0.000	67.5
33		120	22			74	74	67.0	0.5	57.6	1.980	1.980	1.980	1.980	0.72	1.36	1.15	1.00	1.15	30	28	28	1.91	NO	Large factor of safety	0.416	0.000	0.000	0.000	0.000	67.0
33.5		120	22			73	73	66.5	0.5	57.6	2.010	2.010	2.010	2.010	0.71	1.36	1.15	1.00	1.15	30	28	28	1.73	NO	Large factor of safety	0.414	0.000	0.000	0.000	0.000	66.5
34		120	22			73	73	66.0	0.5	57.6	2.040	2.040	2.040	2.040	0.70	1.36	1.15	1.00	1.15	30	28	28	1.59	NO	Large factor of safety	0.413	0.000	0.000	0.000	0.000	66.0
34.5		120	22			73	73	65.5	0.5	57.6	2.070	2.070	2.070	2.070	0.70	1.36	1.15	1.00	1.15	30	28	28	1.48	NO	Large factor of safety	0.411	0.000	0.000	0.000	0.000	65.5
35		120	22			72	72	65.0	0.5	57.6	2.100	2.100	2.100	2.100	0.69	1.36	1.15	1.00	1.15	30	27	27	1.39	NO	Large factor of safety	0.409	0.198	0.012	0.000	0.000	65.0
35.5		120	31			86	86	64.5	0.5	57.6	2.130	2.130	2.130	2.114	0.68	1.36	1.15	1.00	1.15	42	38	38	>4	NO	Large factor of safety	0.410	0.000	0.000	0.000	0.000	64.5
36		120	31			85	85	64.0	0.5	57.6	2.160	2.160	2.160	2.129	0.68	1.36	1.15	1.00	1.15	42	38	38	>4	NO	Large factor of safety	0.411	0.000	0.000	0.000	0.000	64.0
36.5		120	31			85	85	63.5	0.5	57.6	2.190	2.190	2.174	2.143	0.68	1.36	1.15	1.00	1.15	42	38	38	>4	NO	Large factor of safety	0.412	0.000	0.000	0.000	0.000	63.5
37		120	31			85	85	63.0	0.5	57.6	2.220	2.220	2.189	2.158	0.67	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.413	0.000	0.000	0.000	0.000	63.0
37.5		120	31			85	85	62.5	0.5	57.6	2.250	2.250	2.203	2.172	0.67	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.414	0.000	0.000	0.000	0.000	62.5
38		120	31			85	85	62.0	0.5	57.6	2.280	2.280	2.218	2.186	0.67	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.415	0.000	0.000	0.000	0.000	62.0
38.5		120	31			84	84	61.5	0.5	57.6	2.310	2.310	2.232	2.201	0.66	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.415	0.000	0.000	0.000	0.000	61.5
39		120	31			84	84	61.0	0.5	57.6	2.340	2.340	2.246	2.215	0.66	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.416	0.000	0.000	0.000	0.000	61.0
39.5		120	31			84	84	60.5	0.5	57.6	2.370	2.370	2.261	2.230	0.66	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.417	0.000	0.000	0.000	0.000	60.5
40		120	31			84	84	60.0	0.5	57.6	2.400	2.400	2.275	2.244	0.66	1.36	1.15	1.00	1.15	42	37	37	>4	NO	Large factor of safety	0.417	0.000	0.000	0.000	0.000	60.0
40.5		120	23	16		78	78	59.5	0.5	57.6	2.430	2.430	2.290	2.258	0.65	1.36	1.15	1.00	1.15	31	27	31	>4	NO	Large factor of safety	0.418	0.000	0.000	0.000	0.000	59.5
41		120	23	16		77	77	59.0	0.5	57.6	2.460	2.460	2.304	2.273	0.65	1.36	1.15	1.00	1.15	31	27	31	>4	NO	Large factor of safety	0.418	0.000	0.000	0.000	0.000	59.0
41.5		120	23	16		77	77	58.5	0.5	57.6	2.490	2.490	2.318	2.287	0.65	1.36	1.15	1.00	1.15	31	27	31	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	58.5
42		120	23	16		77	77	58.0	0.5	57.6	2.520	2.520	2.333	2.302	0.65	1.36	1.15	1.00	1.15	31	27	31	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	58.0
42.5		120	23	16		77	77	57.5	0.5	57.6	2.550	2.550	2.347	2.316	0.64	1.36	1.15	1.00	1.15	31	27	31	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	57.5
43		120	23	16		77	77	57.0	0.5	57.6	2.580	2.580	2.362	2.330	0.64	1.36	1.15	1.00	1.15	31	26	31	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	57.0
43.5		120	23	16		77	77	56.5	0.5	57.6	2.610	2.610	2.376	2.345	0.64	1.36	1.15	1.00	1.15	31	26	31	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	56.5
44		120	23	16		77	77	56.0	0.5	57.6	2.640	2.640	2.390	2.359	0.64	1.36	1.15	1.00	1.15	31	26	30	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	56.0
44.5		120	23	16		76	76	55.5	0.5	57.6	2.670	2.670	2.405	2.374	0.63	1.36	1.15	1.00	1.15	31	26	30	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	55.5
45		120	23	16		76	76	55.0	0.5	57.6	2.700	2.700	2.419	2.388	0.63	1.36	1.15	1.00	1.15	31	26	30	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	55.0
45.5		120	22	51	19	82	82	54.5	0.5	57.6	2.730	2.730	2.434	2.402	0.63	1.36	1.15	1.00	1.15	30	25	35	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	54.5
46		120	22	51	19	82	82	54.0	0.5	57.6	2.760	2.760	2.448	2.417	0.63	1.36	1.15	1.00	1.15	30	25	35	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	54.0
46.5		120	22	51	19	82	82	53.5	0.5	57.6	2.790	2.790	2.462	2.431	0.62	1.36	1.15	1.00	1.15	30	25	35	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	53.5
47		120	22	51	19	81	81	53.0	0.5	57.6	2.820	2.820	2.477	2.446	0.62	1.36	1.15	1.00	1.15	30	25	34	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	53.0
47.5		120	22	51	19	81	81	52.5	0.5	57.6	2.850	2.850	2.491	2.460	0.62	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.420	0.000	0.000	0.000	0.000	52.5
48		120	22	51	19	81	81	52.0	0.5	57.6	2.880	2.880	2.506	2.474	0.62	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	52.0
48.5		120	22	51	19	81	81	51.5	0.5	57.6	2.910	2.910	2.520	2.489	0.61	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	51.5
49		120	22	51	19	81	81	51.0	0.5	57.6	2.940	2.940	2.534	2.503	0.61	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.419	0.000	0.000	0.000	0.000	51.0
49.5		120	22	51	19	81	81	50.5	0.5	57.6	2.970	2.970	2.549	2.518	0.61	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.418	0.000	0.000	0.000	0.000	50.5
50		120	22	51	19	81	81	50.0	0.5	57.6	3.000	3.000	2.563	2.532	0.61	1.36	1.15	1.00	1.15	30	24	34	>4	NO	Large factor of safety	0.418	0.000	0.000	0.000	0.000	50.0

File No.: 2G-2404001
Project: PS - Moreno Valley

PLATE A-2

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS

INPUT:
Boring No.: B-8
Groundwater Depth: 35.0 feet

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.38
Peak Horiz. Acceleration (g):	0.74

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Classification	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] [Gmax]	From Tbl. 4-4 [geff]	[geff]*100%	From Tbl. 4-5 Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SM	2.5	120.0	0.15	0.10	0.072	12.0	324.427	2.11E-04	1.00E-02	1.00E+00	1.50E+00	14.0524	1.4566	1.75
10.0	5.0	SM	7.5	120.0	0.45	0.30	0.214	27.0	736.330	2.53E-04	1.25E-03	1.25E-01	7.90E-02	14.0524	0.0767	0.09
15.0	5.0	SP-SM	12.5	120.0	0.75	0.50	0.354	18.0	830.424	3.42E-04	1.40E-03	1.40E-01	1.60E-01	14.0524	0.1554	0.19
20.0	5.0	ML	17.5	120.0	1.05	0.70	0.488	86.0	1654.902	2.20E-04	4.90E-04	4.90E-02	1.50E-03	14.0524	0.0015	0.00
25.0	5.0	CL	22.5	120.0	1.35	0.90	0.616	31.0	1335.465	3.23E-04	8.10E-04	8.10E-02	1.00E-04	14.0524	0.0001	0.00
30.0	5.0	SM	27.5	120.0	1.65	1.11	0.735	26.0	1392.338	3.49E-04	8.20E-04	8.20E-02	6.00E-02	14.0524	0.0583	0.07
35.0	5.0	ML	32.5	120.0	1.95	1.31	0.845	30.0	1587.581	3.35E-04	7.70E-04	7.70E-02	4.15E-02	14.0524	0.0403	0.05

** Clay layers not included in the dry sand settlement analysis, unlikely to be affected by seismic ground shakings

Reference: Tokimatsu, K., and Seed H.B. (1987). Evaluation of Settlement in Sands due to Earthquake Shaking." Journal Geotechnical Engineer Division, ASCE, 113 (8), 861-878.

Total Earthquake-Induced Settlements in Dry Sandy Soils (inches) = 2.15
Total Earthquake-Induced Settlements in Dry Sandy Soils (inches) = **0.40**
(By R&R 5-ft)

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of "free" water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an "impervious" material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were "capped" with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles'* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the "Standard Penetration Resistance" or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles'* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as "N". The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled "General Notes".

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are "scanned" in *Giles'* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer's) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or "ash" organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a "sieve analysis," which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a "hydrometer analysis" which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

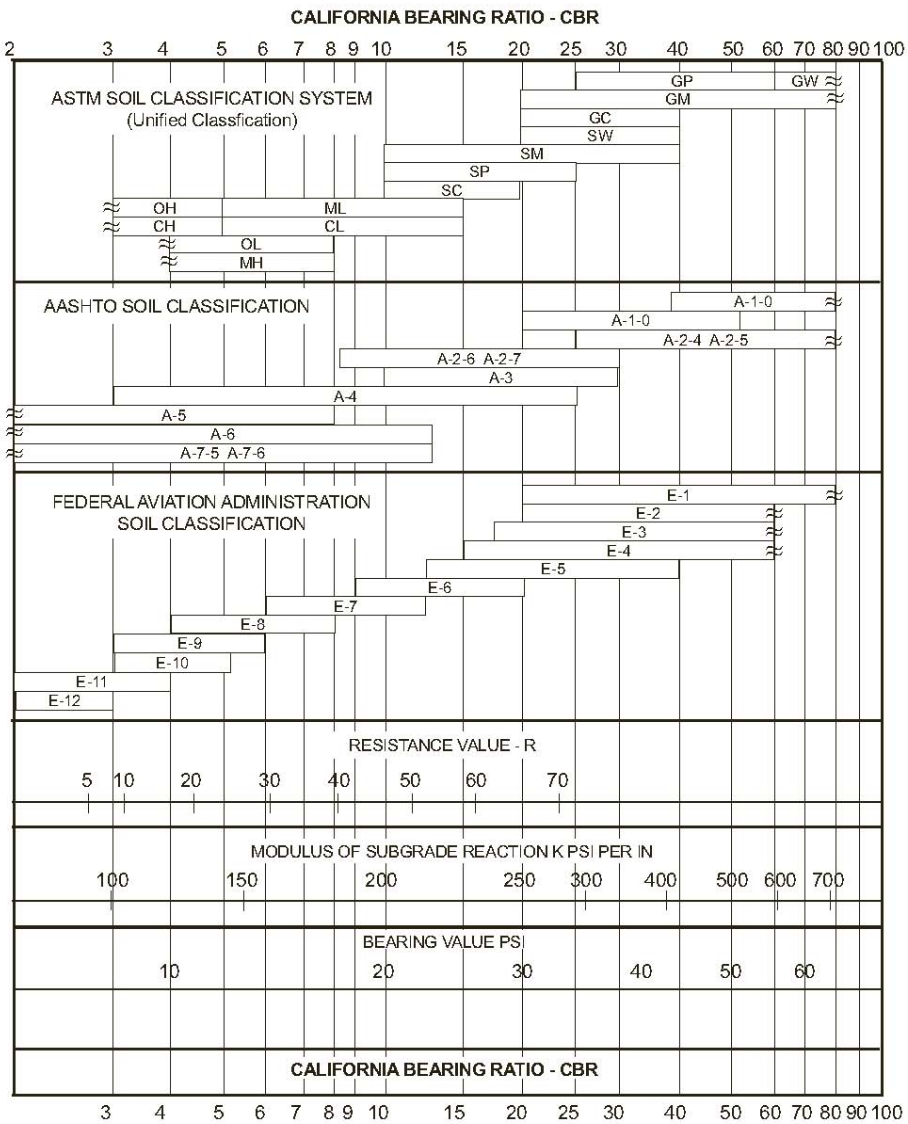
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled "General Notes."

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

GUIDE SPECIFICATIONS FOR SUBGRADE AND PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING MODIFIED PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compacted fill, subgrades, and grades shall be (a) underlain by suitable bearing material, (b) free of all organic frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proofrolling to detect soft, wet, yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar material indicated under Item 5. Note: Compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary for proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement at subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(v) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soils engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated", and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3 inch particle diameter and all underlying compacted fill a maximum 6 inch diameter unless specifically approved by an experienced soils engineer. All fill material must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per Unified Soils Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D-1557) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 95 percent of maximum dry density, or 5 percent higher than underlying structural fill materials. Where the structural fill depth is greater than 20 feet, the portion below 20 feet should have a minimum in-place density of 95 percent of its maximum dry density or 5 percent higher than the top 20 feet. Cohesive soils shall not vary by more than -1 to +3 percent moisture content and granular soil ±3 percent from the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer observing the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a 3±1 percent moisture content above optimum moisture content to limit future heave. Fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavements, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filing, subgrade grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grade/foundation construction must be called to the soils engineer's attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Wherever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work should not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

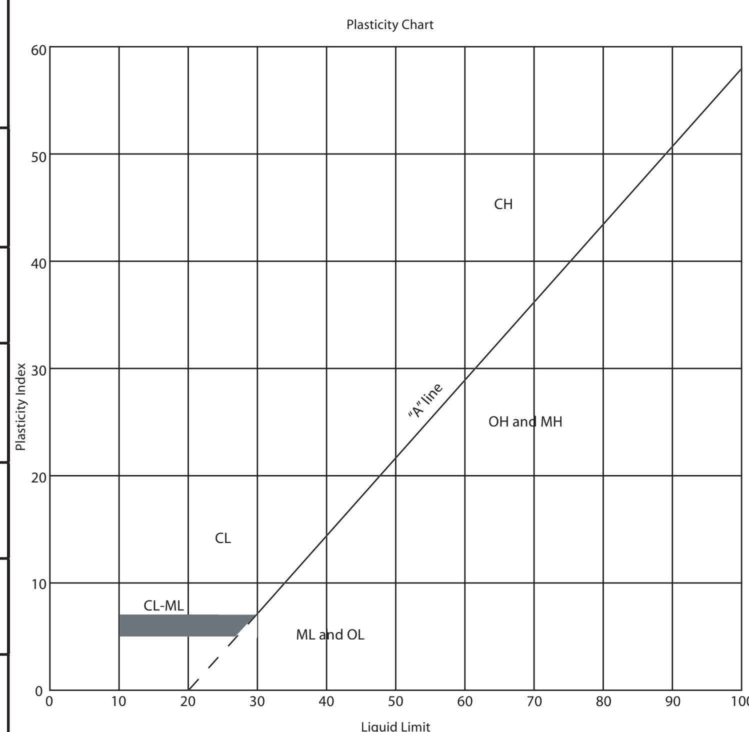
The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *									
Class	**Compaction Characteristics**	**Max. Dry Density Standard Proctor (pcf)**	**Compressibility and Expansion**	**Drainage and Permeability**	**Value as an Embankment Material**	**Value as Subgrade When Not Subject to Frost**	**Value as Base Course**	**Value as Temporary Pavement**	
								With Dust Palliative	**With Bituminous Treatment**
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP; More than 12 percent: GM, GC, SM, SC; 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.
[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	**BLOWS PER FOOT (N)**	**UNCONFINED COMPRESSIVE STRENGTH (TSF)**
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	**BLOWS PER FOOT (N)**
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	**PI**
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	**PI**
Low	0 - 15
Medium	15 - 25
High	25+

Important Information About Your
Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

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

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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