APPENDIX C

Geotechnical Report

● **YELLOW**

Geotechnical Engineering Exploration and Analysis

Proposed New 6-Story Building
Public Storage Facility
5741 West Jefferson Boulevard
Los Angeles, California

Prepared for:

Public Storage
Glendale, California

May 22, 2024
Revised June 10, 2024
Project No. 2G-2009003-R.2

GILES
ENGINEERING ASSOCIATES, INC.

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GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

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

May 22, 2024
Revised June 10, 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 New 6-Story Building
Public Storage Facility
5741 West Jefferson Boulevard
Los Angeles, California
Project No. 2G-2009003-R.2

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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LOS ANGELES, CALIFORNIA
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GEOTECHNICAL ENGINERING EXPLORATION AND ANALYSIS

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PUBLIC STORAGE FACILITY
5741 WEST JEFFERSON BOULEVARD
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PROJECT NO. 2G-2009003-R.2

1.0 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 Hollywood and South ½ Burbank Quadrangles (Dibblee, Jr., 1991)* indicates that the subject site is underlain by young Quaternary alluvial deposits (Qa) consisting mostly of clay, sand and gravel, including gravel and sand of minor stream channels.
- Existing pavement encountered in our test borings consisted of approximately 1 to 3 ½ inches thick asphaltic concrete with 4 to 6 inches of aggregate base noted. Based on our visual observation, the existing pavement is in poor to fair condition.
- Fill soils encountered within our exploratory test borings from about 3 ½ to 15 feet (certified fill to 15 feet in 2 of the 22 borings) with the deepest fill in the central portion of the site, with an average depth of about 3 ½ to 6 feet estimated for the entire site, generally consisted of moist, silty and clayey sand, with a few layers of sandy silt some clay, all determined to be 90 percent or greater, where tested, in regards to the relative density as determined under ASTM D1557. Dry density values were determined from the California Modified ring samples, and compared to the lab results of the Proctors relevant to the appropriate soil type. It is our opinion the fill is adequately compacted and certified fill, as also determined by the geotechnical firm Robert Stone in 1984, in regard to the history of the site, discussed in detail within the report. The data results from Giles field and laboratory relative density testing (dry density/Proctors) are presented with the text of this report and also presented on the borings logs within Appendix A.
- The native soils beneath the fill have a variable and somewhat low strength and moderate to high settlement characteristics, which have been pre-loaded with the existing fill in place at least 40 years. These soils generally consisted of very loose to very dense sand and gravel, and layers of medium stiff to stiff, sandy clay layers, with trace to some organics, with highly variable organic amounts as determined by lab testing generally within the 10 feet through 20 feet below grade (average) range.
- Native soils encountered within our exploratory test borings below about 20 feet generally consisted of damp to very moist, firm to very dense in relative density silty sand and fine to coarse sand with trace clay and gravel with some interbedded layers of firm to stiff in comparative consistency silt and sandy clay.
- Groundwater was not encountered during our subsurface investigation to the maximum depth explored (61.5 feet).

Site Development

- The project includes a new, at-or-near grade, 6-story building, with ground level parking, retail and storage units, and new drive and parking areas. The site is about 2.09 acres, and total square footage of the 6-story structure is approximately 303,453.

Building Foundation

- We recommend the at-grade building be supported by a spread footing or mat foundation in conjunction with ground improvement methods consisting of aggregate or grouted piers, or similar proprietary soil improvement system.
- The mat and/or spread footings should be designed by the structural engineer.
- The ground floor of the new building may be designed as a slab-on-grade and/or load-bearing mat.
- The slab-on-grade or mat will be underlain by a 4-inch thick granular base supported on a properly prepared subgrade.

Pavement

- Asphaltic Concrete: 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 in high stress areas such as entrance/exit aprons lane and in trash enclosure loading zone with a 4 inch granular base.

YELLOW – This site has been given a yellow designation due to increased costs associated with any ground improvement requirements in comparison to traditional geotechnical improvement methods, required in order to strengthen the native clays with organics to depths up to about 20 feet at the very deepest area onsite, for construction of a shallow foundation.

2.0 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 building are provided in this report. Geotechnical-related recommendations are also provided for the proposed parking lot. 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.

3.0 SITES AND PROJECT DESCRIPTION

3.1 Site Description

The subject property, which is currently occupied with several single-story storage buildings, an office, two cellular towers, and parking and drive areas, is located at 5741 West Jefferson Boulevard, in the City of Los Angeles, California (Figure 1). No structural information is available for the existing buildings. Based on visual characteristics, the buildings are assumed to be supported by conventional shallow foundations with a slab-on-grade floor, placed on previously graded and certified fill prior to the reported certified fill in 1984 (Robert Stone & Associates). The site is bounded on the north by commercial buildings, on the east by the relatively new construction of a multi-story apartment building with a two-level basement, on the south by Jefferson Boulevard, and on the west by La Cienega Place. The existing buildings and parking areas are considered to be in fair condition.

Based on a review of *Google Earth Imagery* the site is relatively level (ranging from El. 96 ft to El. 98 ft). The subject property is located at approximately latitude 34.0273° and longitude -118.3748°.

3.1a Site History

Site historical photos were observed on Google Earth and USGS from 1948 to current. It was observed in aerial photos that in 1948 and prior, creeks/sloughs extended through the property at various locations, and were later realigned off site, which is consistent with the random fill depths encountered within our borings, and organic material (likely river deposits) encountered on the site.

Giles researched the available files at the City of Los Angeles archives online and in person to better research and understand the site history in comparison to the aerial photos, in conjunction with our field and laboratory test results. A geotechnical report by Robert Stone & Associates (Robert Stone) on file at the City of Los Angeles was published by Robert Stone for the study of the current soil conditions at that time, and for the design of the now present Public Storage buildings. Within this report, the consultant identified fill similar to the fill that Giles encountered, including the depths and relative compaction (ASTM D1557), indicating in their report that the relative compaction of the fill was determined to be between 90 and 98 percent of the modified proctor per ASTM D1557. In 1985, a compaction report by Robert Stone was published indicating that the upper 3 feet of soil was also compacted as structural fill.

Two additional reports were reviewed by Giles, including reports by Geotechnical Solutions (2004) and Salem (2007) for the cellular towers that were constructed on the eastern area of the site. Geotechnical Solutions reported about 3 feet of fill; Salem did not report fill, but did have similar dry density results as Giles and Robert Stone.

A report from 1960, prepared by Mauseth and Howe Foundation Engineers, available in the City of Los Angeles archives was reviewed, however it was determined from an aerial overlay with our subject property and further research, that this report was for a different site, determined to about 2,000 feet to the southwest of the subject site.

3.2 Proposed Project Description

It is our understanding that the existing buildings will be demolished for the construction of a new, 6-story storage building, with total square footage for the six floors of about 303,453 sq.ft. The proposed building will be an at-or-near grade building without a basement. The proposed ground level will include parking, retail and storage units, and new drive and parking areas. The site is about 2.09 acres. Based on a review of the Conceptual Site Plan dated March 16, 2023, prepared by Ware Malcomb, the proposed building will be set back by approximately 17 feet to 34 feet from the property lines. Proposed parking and drive aisles will be located within the northern portion of the first floor and along the east and north property boundaries.

At the time of this geotechnical investigation, design loads and/or proposed structural conditions were available for our review. Based upon correspondence with the structural engineer the dead load plus live load of 773 kips was determined for the maximum spacing of 30 feet by 30 feet, and 86 kips for the minimum column spacing of 10 feet by 10 feet. The structure is anticipated to be concrete, masonry, and steel.

Preliminary project information did not indicate the planned finished floor elevation for the proposed building. However, it is anticipated that the finish floor elevation of the new building will relatively match the existing grade, with a finish floor elevation of approximately El. 96 to 98. Therefore, site grading is anticipated to include only minor grading in order to establish the necessary site grade to accommodate the assumed floor elevation, exclusive of site preparation requirements necessary to create a stable site suited for the proposed development.

The traffic loading for the proposed parking lot is understood to predominantly consist of automobiles with occasional heavy trucks resulting from deliveries and trash removal. The parking lot pavement sections have been designed on the basis of an assumed Traffic Index of 4.5 for the parking stall areas (light duty) and 5.5 for the drive lanes (medium duty). Pavement designs are based on a 20-year design period.

4.0 SUBSURFACE EXPLORATION

4.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of 22 exploratory test borings to depths of about 21 ½ to 61 ½ feet below existing ground surfaces utilizing a truck mounted hollow stem drill rig, on October 9, 2020, May 20, 2021, and January 3 and 4, 2024. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan, Test Boring Logs (Records of Subsurface Exploration), and test results 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.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials 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. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the borings.

Standard split-spoon tests (SS), also called Standard Penetration Test (SPT), were also 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 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.

4.2 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.

Pavement

Existing pavement encountered in our test borings consisted of approximately 1 to 3 ½ inches thick asphaltic concrete with 4 to 6 aggregate base noted. Based on our visual observation, the existing pavement is in poor to fair condition.

Geology

Our review of the *Geologic Map of the Hollywood and South ½ Burbank Quadrangles (Dibblee, Jr., 1991)* indicates that the subject site is underlain by young Quaternary alluvial deposits (Qa) consisting mostly of clay, sand and gravel, including gravel and sand of minor stream channels.

Soil

Fill soils encountered within our exploratory test borings varied from about 3 ½ to 15 feet, with the deepest fill in the central portion of the site, and with an average, estimated, fill depth of about 3 ½ to 6 feet around it, generally consisted of moist, silty and clayey sand, with a few layers of sandy silt, some clay, all determined to be 90 percent or greater, where tested, in regards to the relative density as determined under ASTM D1557. Dry density values were determined from the California Modified ring samples and compared to the lab results of the Proctors relevant to the appropriate soil type; all samples tested were above 91 percent compaction in regard to the modified proctor. Therefore, it is our opinion the fill is adequately compacted and considered certified fill, similar to the determination by the geotechnical firm Robert Stone in 1984, in regard to the history of the site. The data results from Giles field and laboratory relative density testing (dry density/Proctors) is presented within the text of this report and within the borings logs within Appendix A.

The native soils beneath the fill have a variable and somewhat low strength and moderate to high settlement characteristic. These soils generally consisted of very loose to very dense sand and gravel, and layers of medium stiff to stiff, sandy clay layers, with trace to some organics, generally encountered between about 10 feet through 20 feet below the existing grade (average).

Native soils encountered within our exploratory test borings below about 20 feet generally consisted of damp to very moist, firm to very dense in relative density silty sand and fine to coarse sand with trace clay and gravel with some interbedded layers of firm to stiff in comparative consistency silt and sandy clay.

Groundwater

Groundwater was not encountered during our subsurface investigation to the maximum depth explored (61.5 feet). According to the Hollywood Quadrangle Seismic Hazard Zone Report, the historic high groundwater for the site is about 10 feet below grade.

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.

4.3 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 (P-1 and P-2) 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 (P-1 and P-2) 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 rate was reduced to account for the discharge of water from both the sides and bottom of the boring. The formula given by the County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division was used to calculate for the tested infiltration rate.

Infiltration Rate = Pre-adjusted Percolation Rate divided by Reduction Factor.

Where the reduction factor (Rf) is given by:
Rf = (2di - Δd/ dia) + 1
With:di = initial water depth (in.)
Δd = average/final water level drop (in.)
Dia = diameter of the boring (in.)

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

TABLE 1 – PERCOLATION TEST RESULTS

Test Hole	Test Depth[1] (feet)	Percolation Rate (in/hr)	Infiltration Rate (in/hr)	Soil Type
P-1	26.5	105.6	3.98	Silty Sand
P-2	36.5	252.0	8.09	Silty Sand

1) Depth is referenced to the existing surface grade at the test location.

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

5.0 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 are brief descriptions of our laboratory test results.

Proctor

Three bulk samples representative of the variable fill types previously placed at the site, and within our borings, were sampled, with Proctors performed in our lab in accordance with ASTM Test Method D1557. These proctor values, also presented on our boring logs, are:

Proctor #	Soil Type	Maximum Dry Density (pcf)	Optimum Moisture %
2	Clayey Sand, trace Gravel	121.5	18
2	Silty Sand, some Clay	119.5	12
3	Sandy Silt and Clay	104.5	19

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, and resulted in dry densities ranging between 91 to 99 percent compaction.

Sieve Analysis

Sieve Analyses were performed on selected samples from the test borings to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 422 and ASTM D 1140. The results of the Sieve tests are presented in Test Boring Logs in Appendix A.

Atterberg Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) were determined on selected samples from Test Borings B-1 and B-6 in accordance with Test Method ASTM D 4318. The results of the Atterberg Limits are included on the test borings enclosed in Appendix A.

Consolidation Test

Settlement predictions under anticipated loads were made on the basis of a one-dimensional consolidation test. This test was performed in general conformance with Test Method ASTM D 2435. The test sample was inundated in order to evaluate the sudden increase in moisture condition (collapse/swell potential). The results of the consolidation test are graphically presented as figures in Appendix A.

Expansive Potential

To evaluate the expansive potential of the cohesive soils encountered during our subsurface exploration, a composite sample from Test Borings B-1 to B-6 (3 ½ and 5 feet) was subjected to an Expansive Index (EI) test in accordance with Test Method ASTM D 4829. The result of our EI test indicates that the near surface, composite sample has a *very low* expansion potential (EI = 0).

Organic Content

The organic content was determined for representative samples of the on-site soils in accordance with Test Method ASTM D 2974. The results of these tests are presented on the boring logs within Appendix A.

6.0 GEOLOGIC AND SEISMIC HAZARDS

6.1 Active Fault Zones

The project site is located in the highly seismic Southern California region within the influence of several fault systems. However, the site does not lie within the boundaries of an Earthquake Fault Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong ground shaking during seismic activity.

6.2 Seismic Hazard Zones

According to the Seismic Hazard Zone report for the Hollywood Quadrangle published by the CGS, the site is located within a liquefaction hazard zone. Additionally, as noted within the Seismic Hazard Zone Report, the historic high groundwater is anticipated to be about 10 feet below grade. Therefore, liquefaction analysis is deemed necessary for this site.

General types of ground failures that might occur as a consequence of severe ground shaking typically include ground lurching and shallow ground rupture. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on our subsurface exploration and the seismic designation for this site, all of the above effects of ground failure due to seismic activity are considered unlikely at the site.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration and laboratory testing, development for the subject site with the proposed project is considered feasible from a geotechnical point of view provided the following conclusions and recommendations are incorporated in the design and project specifications.

Conditions that may result from the proposed improvement have been evaluated on the basis of the engineering characteristics of the subsurface materials encountered during our subsurface investigation and their anticipated behavior both during and after construction. Conclusions and recommendations, along with site preparation recommendations and construction considerations are discussed in the following sections of this report.

We recommend that Giles Engineering Associates, Inc. be involved in the review of the grading and foundation plans for the site. Based on the results of our review, modifications to our recommendations 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 city code and in accordance with the recommendations presented herein.

7.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

The site is not located within the Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong ground shaking during seismic activity. 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 2022 California Building Code (CBC), the Newport-Inglewood Connected alt 2 and Newport-Inglewood, alt 1 faults are the closest known active faults and are located approximately 0.25 and 0.36 miles from the site, respectively. Based upon a deaggregation analysis the Maximum Magnitude (Mw) earthquake is 7.5.

The proposed structure should be designed in accordance with the current versions of the: *2022 California Building Code (CBC)*, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures ASCE 7-16*, and applicable local codes.

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)	2.001
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.709
Site Coefficient, F_a short period	1.0
Site Coefficient, F_v 1-second period	1.7
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS}	2.001
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1}	1.205
Design Spectral Response Acceleration Parameter, S_{DS}	1.334
Design Spectral Response Acceleration Parameter, S_{D1}	0.803
MCE_G Peak Ground Acceleration adjusted for site class effects, PGA_M	0.944

According to Section 11.4.7 of ASCE 7-16, 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) may 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 $T_L \geq$ 1.5Ts, or Equation (12.8-4) for $T > T_L$.

Liquefaction

The *Seismic Hazard Zone Report for the Hollywood Quadrangle* published by the California Geological Survey (CGS) indicates this site is within a zone that is potentially liquefiable and therefore requires a liquefaction study. Accordingly, a detailed liquefaction analysis was performed.

The liquefaction analysis was performed utilizing the computer software program LiquefyPro and based on the 2022 CBC, ASCE 7-16, California Geological Survey (CGS) Special Publication 117A, and additional City of Los Angeles requirements for liquefaction analysis. For the 475- and 2,475-year return periods we used variables of ⅔ PGA_M, and PGA_M (Design Level Earthquake, and Maximum Considered Earthquake [MCE], respectively [ASCE 7-16, CH11.5]) and a factor of safety of 1.1, or 1.0, respectively, in the determination of liquefiable soils. The liquefiable results are presented graphically in Appendix A with the computer output files.

For this analysis we used the soil profiles identified within Borings B-1 and B-6, the historic high groundwater depth of 10 feet, and site accelerations (PGA_M and ⅔ PGA_M) of 0.944g and 0.629g, as obtained from the USGS web site and determined from ASCE 7-16. Corresponding site moment magnitudes of 6.51 and 6.5 were determined using Deaggregation methods published by USGS for the 475- and 2,475-year return periods, respectively. Values used in our liquefaction analysis are also included in Appendix A with the liquefaction calculations and results.

The on-site, fine-grained soils (clay and silt) were evaluated to determine susceptibility to liquefaction during ground shaking in accordance with the criteria outlined within the California Geological Survey (CGS) Special Publication 117A and CLA requirements. Soils considered to be potentially susceptible to undergo seismically induced deformation during liquefaction are classified in the following manner:

1. Plastic Index (PI) < 12 and moisture content greater than 85 percent of the Liquid Limit
2. Sensitive soils with PI > 18.
3. All loose to medium dense granular soils.

The soils obtained during our subsurface exploration, which were considered to possibly undergo seismically induced deformation, were tested for CGS Special Publication 117A guidelines with results in the table below.

Test Boring No. & Depth	Liquid Limit (LL)	Plastic Index (PI)	In-situ Moisture	W_c/LL
B-1 @ 50 ft. [1]	38	5	23	0.61

[1] Non-liquefiable

The results of our analysis with a high water table of 10 feet indicate that the site soils are susceptible to the following soil liquefaction magnitudes.

Liquefaction (inches)		
Boring	10% in 50 yr	2% in 50 yr
B-1	1.28	1.64
B-6	0.93	1.57

The liquefaction analysis was performed using the computer program LiquefyPro (Version 5) developed by Civil Tech Software. The program is based on the most recent publications of the NCEER Workshop and SP117 Implementation. Corrected SPT blow counts based upon hammer energy ratio, borehole diameter and sampling method were used in analysis calculations. The liquefiable results are presented graphically in Appendix A with the computer output files.

In accordance with the City of Los Angeles Bulletin 2020 Los Angeles Building Code and Information Bulletins, dated January 9, 2020, and seismically induced settlements required to be evaluated based upon ASCE 7-16 Sections 11.8 and 12.13.9, the calculated maximum differential settlement allowed per Table 12.13-3 Differential Settlement Threshold is 0.010(L), where L is considered to be the maximum spacing from column to column, or 30 feet. Therefore, the maximum allowable differential settlement is 3.6 inches (0.010 x 30 feet) over the 30-foot span, for Differential Settlement Threshold, which is greater than the calculated 0.83 inches. Therefore, according to ASCE 7-16 Section 12.13.9.2 design beyond the requirements of Section 12.13.9.2.1 i.e. deep foundation is not required due to seismic. The above liquefaction values are based upon unimproved soil conditions and therefore, depending on the method of geotechnical site improvements the liquefaction analyses should be evaluated based on the proposed conditions, and will lower the values based on soil improvements.

Additionally, as specified by ASCE 11.5 Commentary:

> *The first basis for seismic design in the standard is that structures have a suitably low likelihood of **collapse** in the rare events defined as maximum considered earthquakes (MCE) ground motion [Full PGA; 2% in 50 year]. A second basis is that life threatening damage, primarily from failure of non-structural components in and on structures, is unlikely in a design earthquake ground motion (defined as 2/3rds of the MCE; [10% in 50 year]).*

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 extent on ground surfaces with a very gentle slope. Based upon our liquefaction and magnitudes analysis, the potential for lateral spreading in our opinion is considered to be low.

Liquefaction–Induced Potential for Surface Manifestation

Based on our review of the relationships between the thickness of potentially liquefiable soil layers relative to the thickness of non-liquefiable soil layers developed by Ishihara (1985), it is our opinion that the potential for surface manifestations (sand boils, loss of bearing, etc.) resulting from soil liquefaction at this site is very low.

7.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations.

Site Clearing and Demolition

All structural materials associated with the existing building, including footings and floor slabs, should be removed from the site. Clearing operations should also include the removal of all existing structural features such as asphaltic concrete pavement, and concrete walkways within the area of the proposed new building and site improvements. Existing pavement within areas of proposed development should be removed or processed to a maximum 3-inch size and stockpiled for use as compacted fill or stabilizing material for the new development. Processed asphalt may be used as fill, sub-base course material, or subgrade stabilization material beyond the building perimeter. Processed concrete or existing base may be used as fill, sub-base course material, or subgrade stabilization material both within and outside of the building perimeter. Clean existing base may be reused as base for the new pavement. Due to the moisture sensitivity and variable support characteristics of the on-site soils, the pavement is recommended to remain in-place as long as possible to help protect the subgrade from construction traffic disturbance. Reuse of processed asphalt and/or concrete as recommended, is also contingent upon agency approval.

Should any unusual soil conditions or subsurface structures be encountered during 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 will be preserved are recommended to be relocated outside the building area. Utilities that are not to be reused should be capped off at the property boundary and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations made for removed utilities 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, grading operations must be carefully performed so as not to disturb or damage the existing utility.

New Building Area

Due to the clayey native soil with trace organics and estimated seismic, total settlements of about 1.9 inches, based upon the column loads at a 30 feet by 30 feet spacing; the variable thicknesses of the clay layers; and variable column loads (10 feet by 10 feet, 10 feet by 20 feet, and 30 feet by 30 feet), ground improvements including acceptable aggregate and/or grouted ground improvement elements are recommended, resulting in a combined ground improvement with conventional foundation system, or mat foundation, if desired. Ground improvements are typically a proprietary system. Subgrade preparation and resulting foundation recommendations are therefore to be determined by the ground improvement specialist. However, a spread footing foundation and slab-on-grade floor are expected to be suitable.

We expect that the ground improvements will extend from depths of about 1 foot to about 20 feet, in order to create a uniform, stable foundation subgrade for all foundation systems. All ground improvement design should be reviewed by Giles as the Geotechnical Engineer of Record.

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 trench backfill should be placed in lifts no greater than 12 inches in thickness, moisture conditioned 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, probe, and test the backfills to document adequacy of compaction.

Proofroll and Compact Subgrade

Following lowering of site grades where necessary, the subgrades within the proposed pavement 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. Following proofrolling and completion of any necessary over-excavation, the subgrades where determined necessary by the geotechnical engineer should be scarified to a minimum depth of 8 inches, moisture conditioned 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. 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 very low expansive materials may be reused as structural compacted fill provided they do not contain oversized materials and significant quantities of organic matter or other deleterious materials. Due to the moisture sensitivity of the site soils, care should be used in controlling the moisture content of the soils to achieve proper compaction for load bearing and pavement support. 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 based on the conditions encountered. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations or over the slope to the east. 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

Material for engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. On-site excavated soils that meet these requirements may be used to backfill the excavated building pad and pavement areas.

All fill should be placed in 8-inch-thick maximum loose lifts, water or air dried and then compacted to at least 90 percent 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 expansive (EI < 21) 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.

7.3 Construction Considerations

Soil Excavation

Some slope stability problems should be expected 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. Shallow excavations may be adequately sloped for bank stability while deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

Earthwork (cut/fill) has been estimated by the project civil team to be up to 15,000 CY if considering fill removal up to an average over excavation depth of about 8 to 9 feet, below existing grade, within the first-floor footprint. If this removal and recompaction (R&R) is pursued, Giles will provide recommendations for the most feasible and economical foundation types and bearing capacity with corresponding settlements (total and differential), and other recommendations, for an R&R improvement method.

7.4 Foundation Recommendations

It is recommended that Giles review the foundation and grading plans for the site and if warranted provide recommendations as deemed appropriate.

Ground Improvements: The proposed structure is expected to be designed as a conventional foundation system supported by ground improvement elements estimated on a lateral grid spacing of 8 feet by 8 feet in each direction for a mat, or more concentrated in the areas of spread footings, from depths of about 1 to 20 feet below grade, with spacing expected to increase or decrease depending on various concentrated loads, soils, and other variables. We recommend that the ground improvement elements extend through the existing fill and into competent dense soils, estimated at 25 feet, to provide a uniform subgrade and foundation bearing soil at the bottom of the foundation elevations.

Foundations are expected to be designed based on a net, allowable bearing pressure determined by the ground improvement engineer (estimated to be 4,000 to 6,000 psf for a convention foundation system, and a Modulus of Subgrade Reaction (Ks) of about 85 to 125 psi/in for a mat foundation, based on similar projects) and verified based upon a field proof load testing program. A one-third increase in the allowable soil bearing pressure is expected to be suitable for short-term wind and/or seismic loads. However, as previously indicated, ground improvements are proprietary systems and final recommendations are determined by the ground improvements specialist.

Reinforcing

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

Lateral Load Resistance

Lateral resistance for should be determined by the ground improvement specialist following final designs.

Bearing Material Criteria

For design and construction estimating purposes, suitable bearing soils are expected to be encountered at nominal foundation depths following the recommended site preparation activities. However, proof load testing by the Ground Improvement specialist and field testing by the Geotechnical Engineer within the foundation bearing soils during construction is recommended to document that the foundation support soils possess the minimum strength parameters determined by the Ground Improvement firm.

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

For a mat or conventional foundation for the building supported by ground improvements, the total static and seismic settlement should be determined by the ground improvement specialist for their proprietary system, but is expected to be less 1 inch, with a differential settlement less than ½ inch in 30 feet for a conventional foundation, and ½ inch in 40 feet for a mat foundation.

7.5 Mat and/or Slab-on-Grade Recommendations

Subgrade

The foundation subgrade should be prepared in accordance with the appropriate recommendations presented in the Site Development Recommendations section of this report. Foundation, utility trenches and other below-mat excavations should be backfilled with structural compacted fill in accordance with the project specifications.

Design

The ground floor of the proposed structure may be designed as load-bearing mat based on the recommendations presented in the foundation section of this report or slab-on-grade.

The mat or slab-on-grade is recommended to be underlain by a 4-inch thick, free-draining granular base. A minimum 15-mil synthetic sheet should be provided to serve as a vapor barrier. The vapor barrier is recommended to be in accordance with ASTM E 1745-97, 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 and/or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand may be needed between the mat or slab concrete and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and concrete curling.

7.6 Elevator Pit Recommendations

The proposed 6-story building will include elevator pits. Details and locations of the elevator pits were not provided; however, it is assumed that the pits to be reinforced cast-in-place concrete structures structurally connected to the adjacent mat and/or spread footings, extending about 5 feet below the elevation of the lowest level floor. Giles must be notified if elevator pits will be deeper; this report may need to be revised. Geotechnical-related recommendations for elevator pits that are 5 feet deep are provided below.

This report assumes that elevator pits will be 5 feet below the ground floor. Therefore, the floor of the elevator pits will be supported by suitable bearing soil. The bearing pressure of the mat is therefore recommended for the elevator pit bearing pressure. It is assumed that elevator pit walls will be cast against existing soil, or that engineered fill between the walls and surrounding soil will consist or properly compacted, imported granular material approved by the geotechnical engineer. Based on that assumption, elevator walls are to be design for an "at-rest" equivalent fluid pressure of 55 pcf. Horizontal pressures caused by surface and subsurface surcharge loads must be added to the "at-rest" fluid pressure. Giles will provide supplemental recommendations on a case-by-case basis, but would require specific structural information.

7.7 Construction Considerations

Construction Dewatering

As mentioned previously, groundwater was not encountered during our subsurface investigation. Therefore, groundwater is not expected to impact excavations for the foundation and utilities. However, the site may be susceptible to the development of shallow perched water conditions. In the event that shallow perched water is encountered, filter sump pumps placed within pits in the bottoms of excavations are expected to be the most feasible method of construction dewatering.

7.8 Retaining Walls

Due to the existing site grades and planned building layout, it is possible that retaining walls may be required.

Retaining wall(s) may be designed such as conventional reinforced concrete cantilevered walls supported by spread footings designed for an allowable soil bearing pressures of 2,500 psf.

Static and Seismic Lateral Earth Pressures

Retaining wall design and parameters were determined in accordance with LADBS Information Bulletin, Retaining Wall Design, dated January 1, 2020. Although unlikely, any walls that will exceed 6 feet in height, and considering a Site Class D, are required to include seismic forces with static forces for the design of earth pressures on the permanent walls. A representative phi angle of 33 was correlated for all soil conditions to compute the lateral earth equivalent fluid pressures.

For at-rest (static) conditions, an equivalent fluid pressure (EFP) of 55 pcf is recommended, for a retaining structure. For Active conditions an EFP of 35 pcf is recommended. For active conditions and walls over 6 feet, the total lateral earth pressure (active pressure plus the seismic increment of the lateral earth pressure), results in 35 pcf plus 28 pcf, respectively, for a total of 63 pcf (EFP). The seismic increase was determined per LABDS Bulletin January 1, 2020, using the seismic coefficient (Kh), which was ½ of ⅔ the maximum PGA (PGA_m = 0.944g) for the site, and ¾ of Kh, with PGAm determined from the USGS Design Maps tool from the USGS website.

The above pressures also consider level backfill extending at least 150% of the wall height behind the retaining structure and surface drainage directed away from the wall. Backfill behind the retaining structure should consist of free-draining granular materials. The EFP above may be used for on-site soils to be used as backfill materials. Imported soils (EI < 21) and/or backfill being placed behind the retaining structure should be tested by the geotechnical engineer prior to placement to verify strength and drainage parameters. All retaining walls should be designed with a proper subdrain system. All walls should also be designed to support any adjacent structural surcharge loads imposed vehicle or structural loading, in addition to the above recommended active earth pressure.

Undrained conditions during construction may occur behind the retaining wall and therefore these structures must be designed for undrained earth pressures. For the undrained active pressures and for undrained at-rest pressures EFPs of 80 and 90 are recommended, respectively.

For the design of a retaining system, an ultimate lateral bearing value (passive value) of 400 pounds per square foot per foot of depth may be assumed for soils below the level of excavation. However, passive resistance should be ignored within the upper foot due to possible disturbance. The construction of the retaining system should be monitored continuously and any adjacent structures should be observed for any potential lateral and vertical movement. Plans for the retaining structures should be reviewed by Giles once available.

Drainage and Damp-proofing

Retaining walls are recommended to be designed for drained earth pressures where possible, and therefore, adequate drainage should be provided behind the walls. This can be accomplished by installing subdrains at the base of the walls. Wall footing-drains should consist of a system of filter material and perforated pipe. The perforated pipe system should consist of 4-inch diameter, schedule 40, PVC pipe or equivalent, embedded in ¾-inch open graded gravel or crushed rock enveloped in Mirafi 140 geofabric or equivalent. The pipe should be placed at the base of the wall, and then routed to a suitable area for discharge of accumulated water.

Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Damp-proofing of walls below-grade is recommended to prevent efflorescence.

Wall Backfill

Retaining structure backfill behind the drainage layers should consist of low-expansive on-site or imported soils (EI < 21), as determined by ASTM D 4829 method, and approved by the geotechnical engineer. Retaining structure backfill should not contain organic material, rubble, debris, and rocks or cemented fragments larger than 3 inches in greatest dimension. A 1-foot thick, low-expansive cohesive layer or pavement should be placed at the surface to help prevent surface water intrusion. A geotextile or filter fabric should be placed between the granular drainage layers and adjacent soils (excavated face or compacted materials) to prevent fines from migrating into the drainage layers.

Backfill should be placed in lifts not exceeding 8 inches in thickness, moisture conditioned as necessary, and mechanically compacted throughout to at least 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D 1557). Retaining walls should be properly braced prior to placement and compaction of backfill should be performed with extreme care not to damage the walls.

Elevator pits should be designed as watertight structures with undrained lateral soil pressure, or as a drained condition with sub-drain system stepping down to below the elevator pits.

7.9 Pavement Recommendations

New Pavement Subgrades

Pavement is expected to have reduced service life as a result of deep, in-situ fill. The subgrade in areas of new pavement construction is expected to consist of soils that exhibit a very low to low expansion potential. However, the anticipated subgrade soils are estimated to possess an R-value of 40 to 50. An R-value of 40 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of Los Angeles 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 12 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.

7.10 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.

7.11 Basis of Report

The actual services for the project varied somewhat from those described in the proposed scopes of work, due to 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.

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) EXISTING FEATURES DEVELOPED FROM THE "SITE DEMOLITION PLAN" (SHEET A1.1), DATED 9-13-2023, PROVIDED BY THE CLIENT.

3.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "CONCEPTUAL SITE PLAN" (SHEET A1.0), DATED 9-13-2023, PROVIDED BY THE CLIENT.

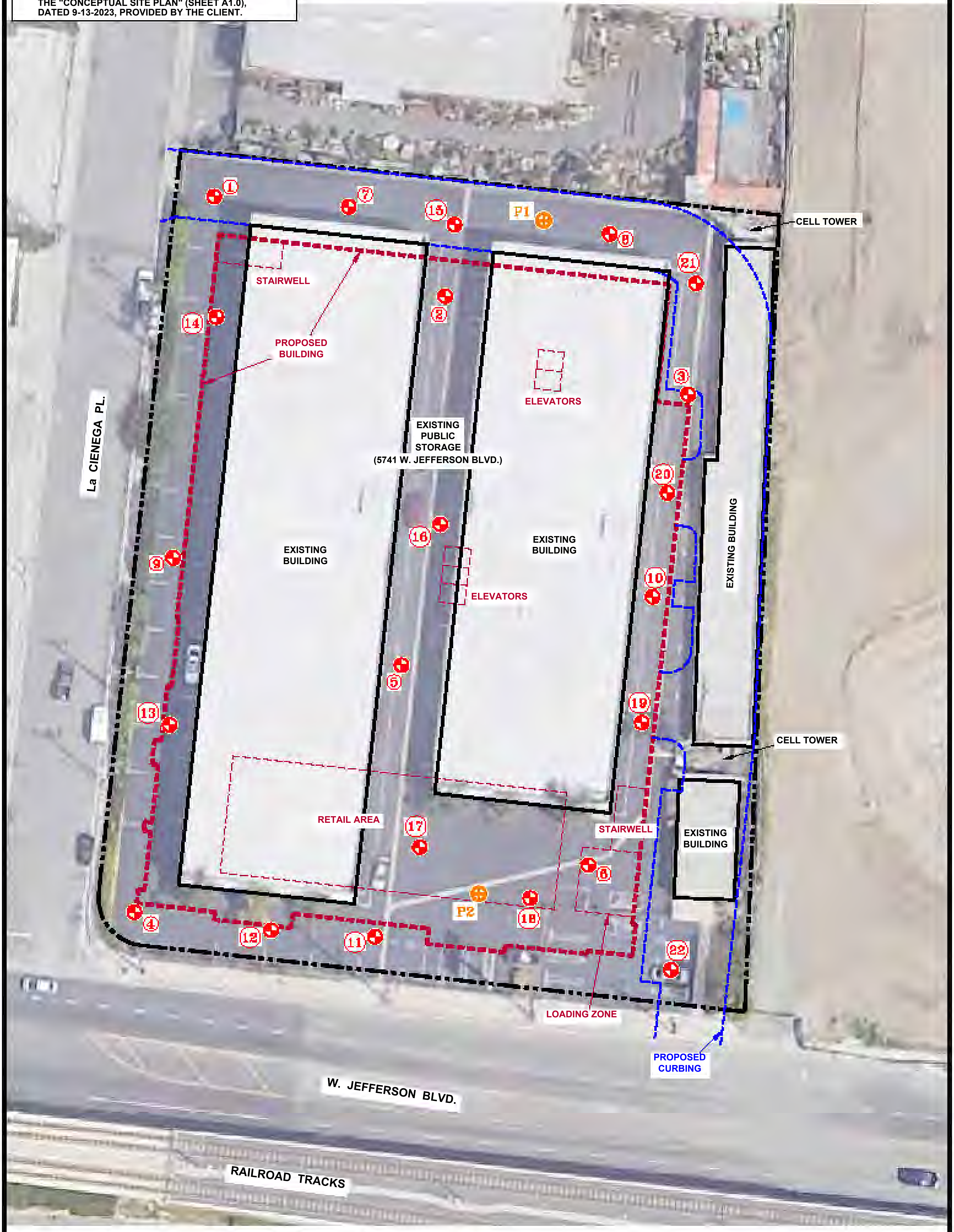

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FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED NEW 6-STORY BUILDING
PUBLIC STORAGE FACILITY
5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
WML/JLM		approx. 1"=40'	01-02-24	03-08-24

PROJECT NO.: 2G-2009003

CAD No. 2g2009003-blp4

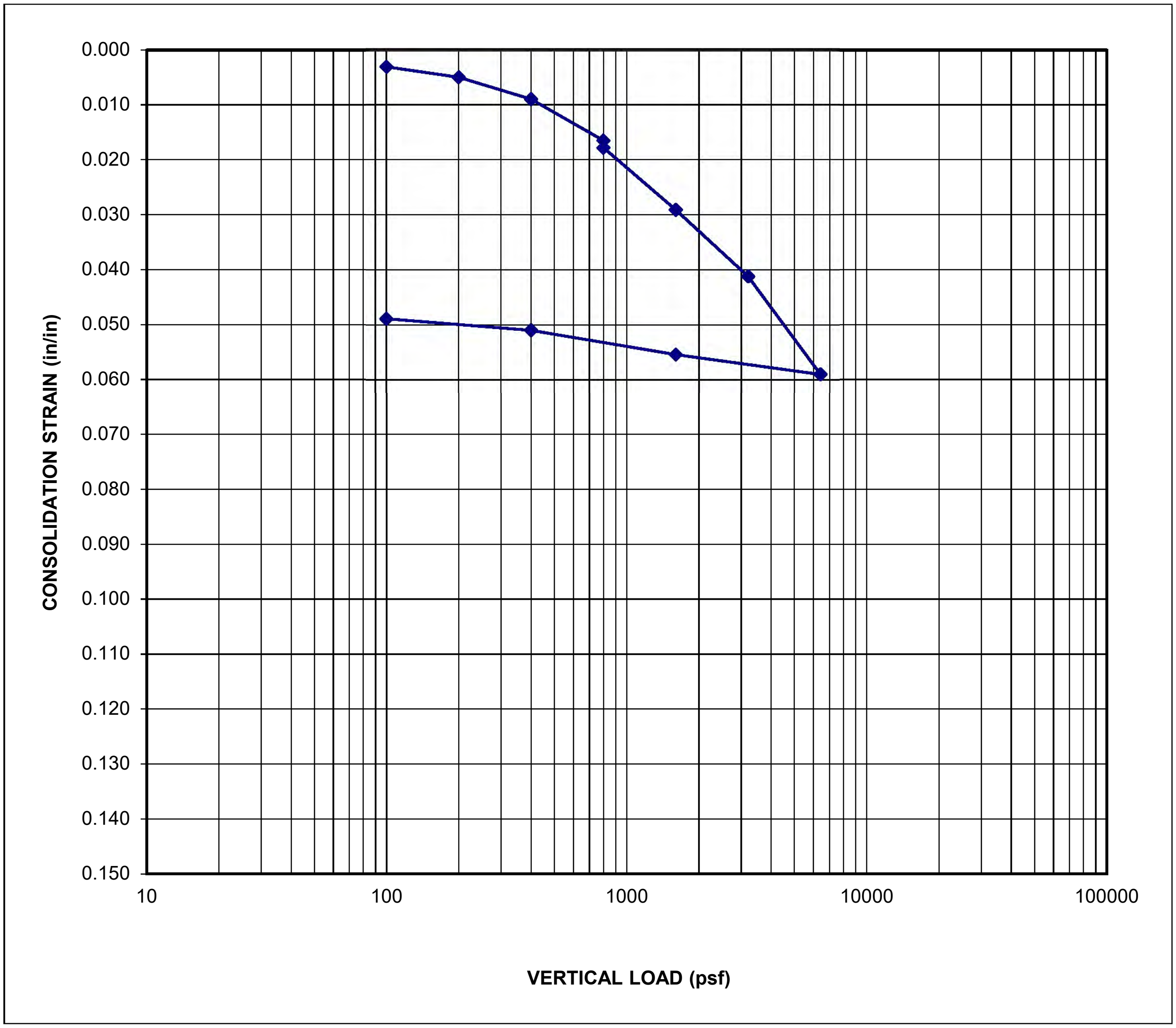
N
W E
S

0 20' 40'
APPROXIMATE SCALE

LEGEND:

1–22	GEOTECHNICAL TEST BORINGS
P1	PERCOLATION TEST BORING

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(ML) - Silt, trace Clay, little organics		
Boring No.	B-14		
Sample No.	4-CS	Initial Moisture Content (%)	38.0
Depth (ft.)	10 ft	Final Moisture Content (%)	38.9
Elevation (ft.)	82	Natural Density (pcf)	107.4
Liquid Limit	N/A	Initial Dry Density (pcf)	74.6
Plastic Limit	N/A	Final Dry Density (pcf)	74.7
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.13%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project: Public Storage
Los Angeles
Client: Public Storage

Project No.: 2G-2009003

Figure No.: **2**

GILES ENGINEERING ASSOCIATES, INC.

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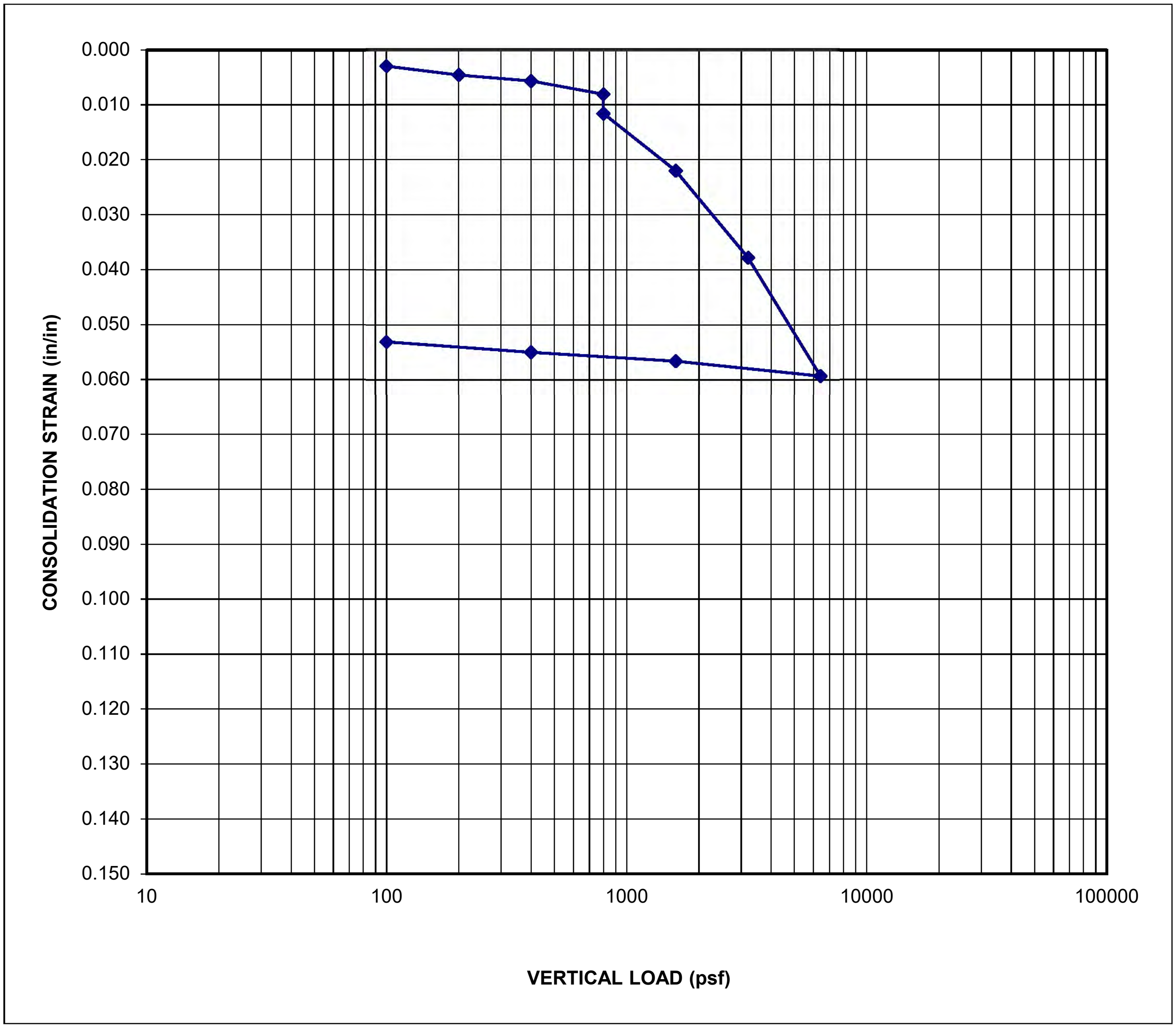
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SC) - Clayey Sand		
Boring No.	B-16		
Sample No.	4-CS	Initial Moisture Content (%)	13.0
Depth (ft.)	10 ft	Final Moisture Content (%)	23.6
Elevation (ft.)	83	Natural Density (pcf)	113.1
Liquid Limit	N/A	Initial Dry Density (pcf)	98.2
Plastic Limit	N/A	Final Dry Density (pcf)	98.5
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.35%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project: Public Storage
Los Angeles
Client: Public Storage

Project No.: 2G-2009003

Figure No.: **3**

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
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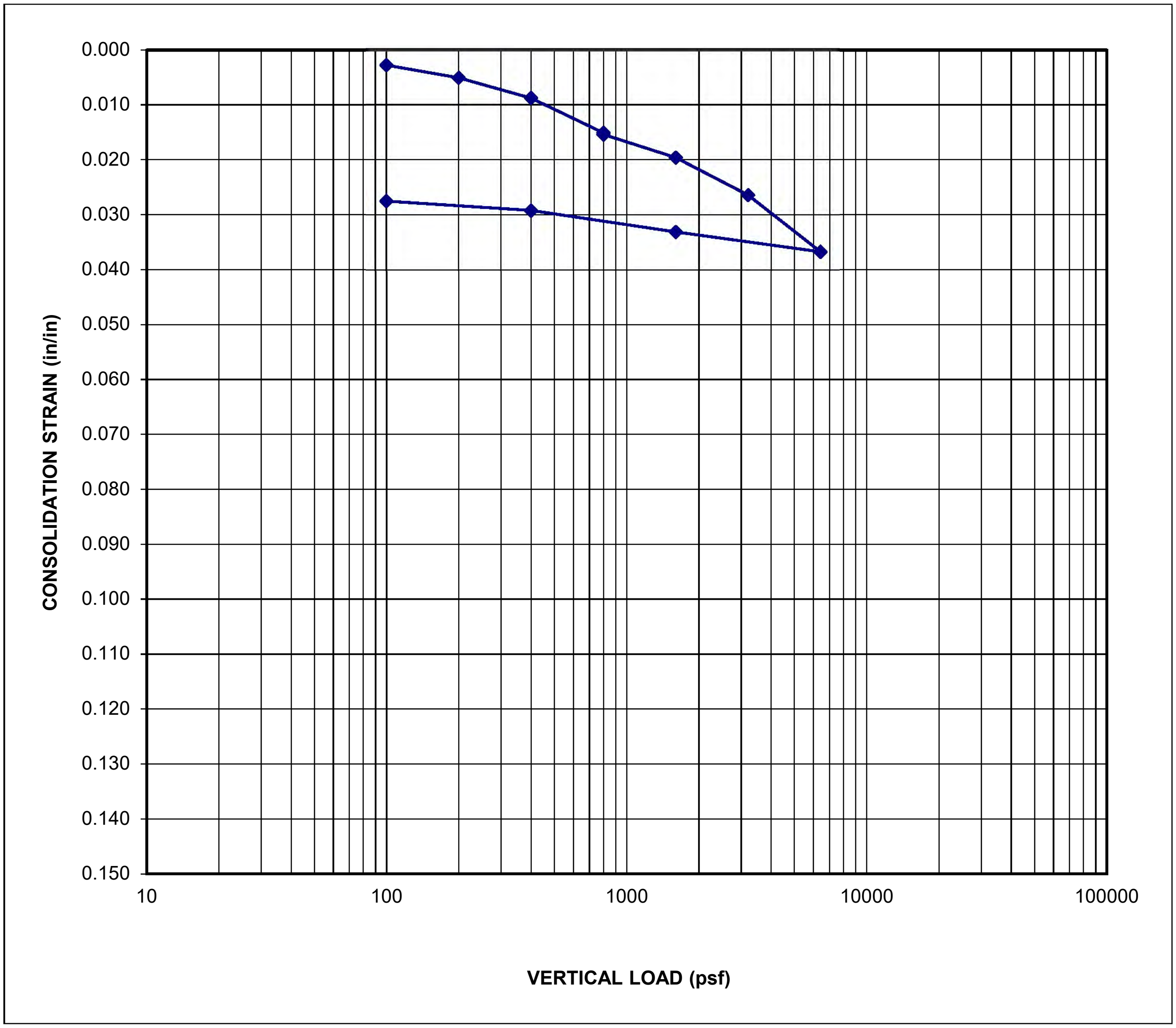

Classification	(ML) - Sandy Silt, trace to little organics		
Boring No.	B-16		
Sample No.	5-CS	Initial Moisture Content (%)	39.0
Depth (ft.)	15 ft	Final Moisture Content (%)	38.5
Elevation (ft.)	78	Natural Density (pcf)	111.2
Liquid Limit	N/A	Initial Dry Density (pcf)	74.9
Plastic Limit	N/A	Final Dry Density (pcf)	74.9
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
Los Angeles
Client: Public Storage

Project No.: 2G-2009003

Figure No.: **4**

GILES ENGINEERING ASSOCIATES, INC.

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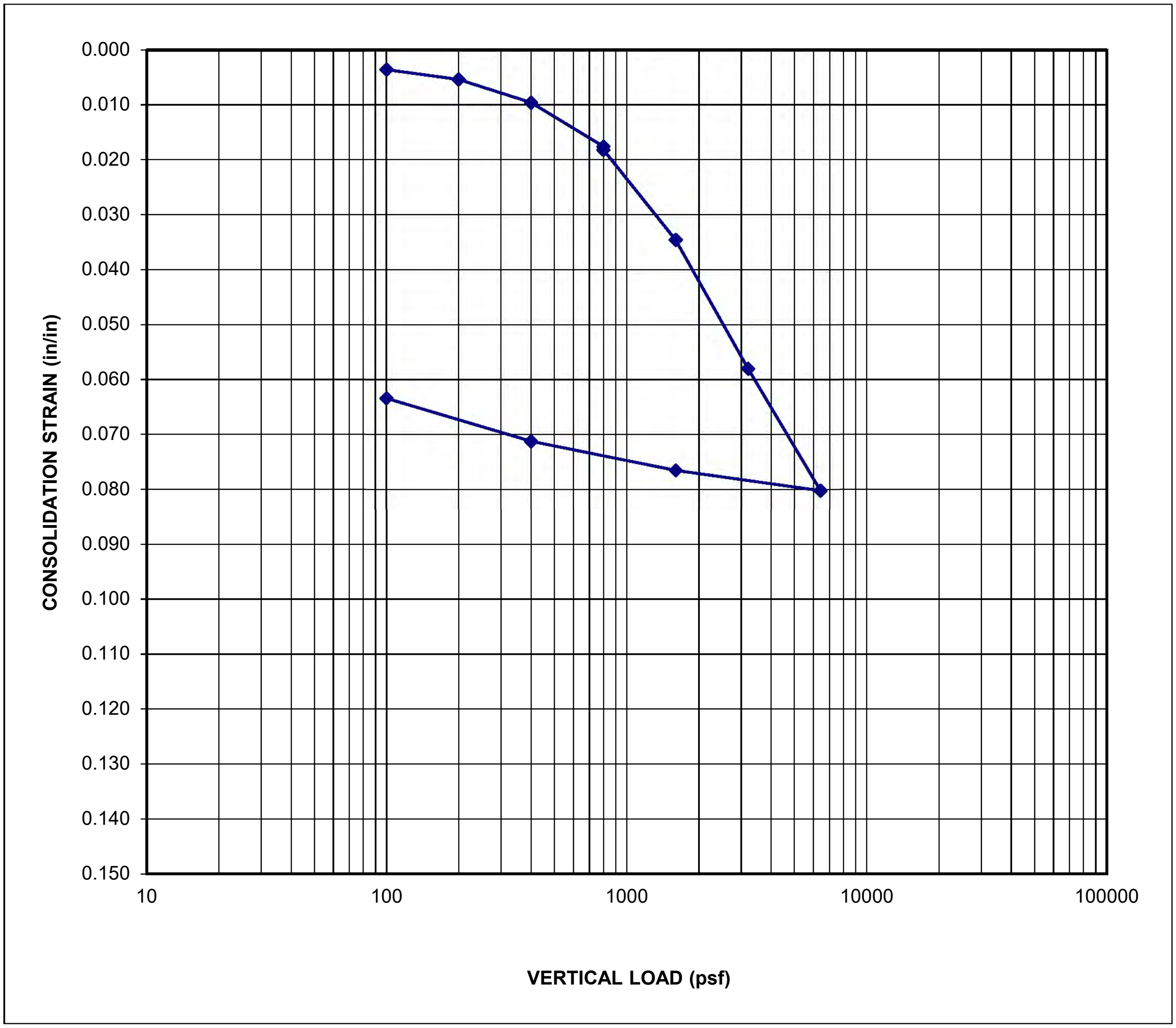
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(CL) Silty Clay		
Boring No.	**B-17**		
Sample No.	3-CS	Initial Moisture Content (%)	40.0
Depth (ft.)	**6 ft**	Final Moisture Content (%)	42.6
Elevation (ft.)	87	Natural Density (pcf)	113.9
Liquid Limit	N/A	Initial Dry Density (pcf)	80.0
Plastic Limit	N/A	Final Dry Density (pcf)	80.0
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.06%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project:	Public Storage Los Angeles
Client:	Public Storage
Project No.:	2G-2009003
Figure No.:	**5**

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
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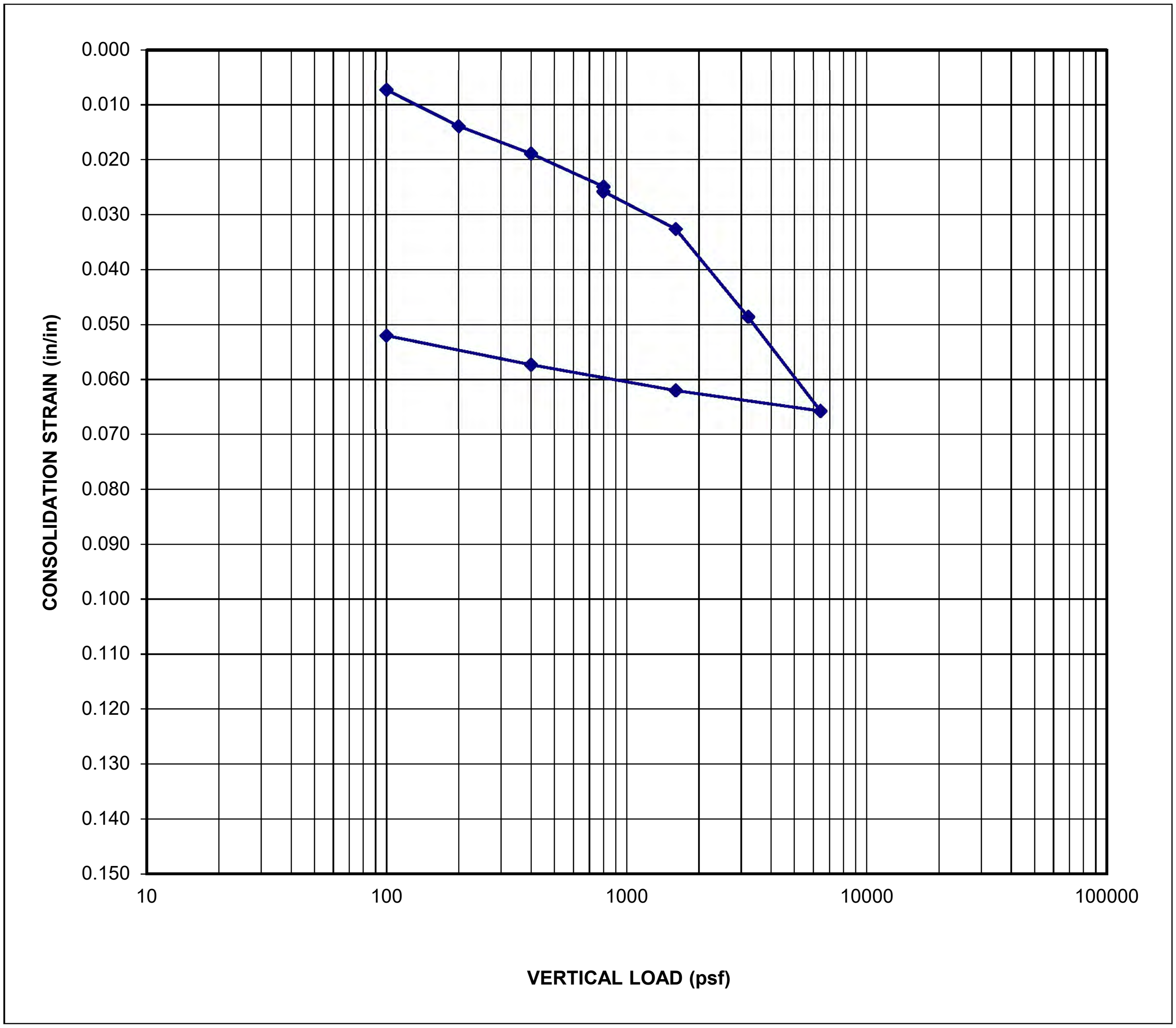
OFFICE: 714-279-0817 FAX : 714-279-9687

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification (ML) - Sandy Silt, fine to medium Sand, little to some organics

Boring No.	**B-19**		
Sample No.	5-CS	Initial Moisture Content (%)	63.0
Depth (ft.)	**15 ft**	Final Moisture Content (%)	69.1
Elevation (ft.)	78	Natural Density (pcf)	119.3
Liquid Limit	N/A	Initial Dry Density (pcf)	73.2
Plastic Limit	N/A	Final Dry Density (pcf)	80.4
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.09%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project: Public Storage
Los Angeles
Client: Public Storage

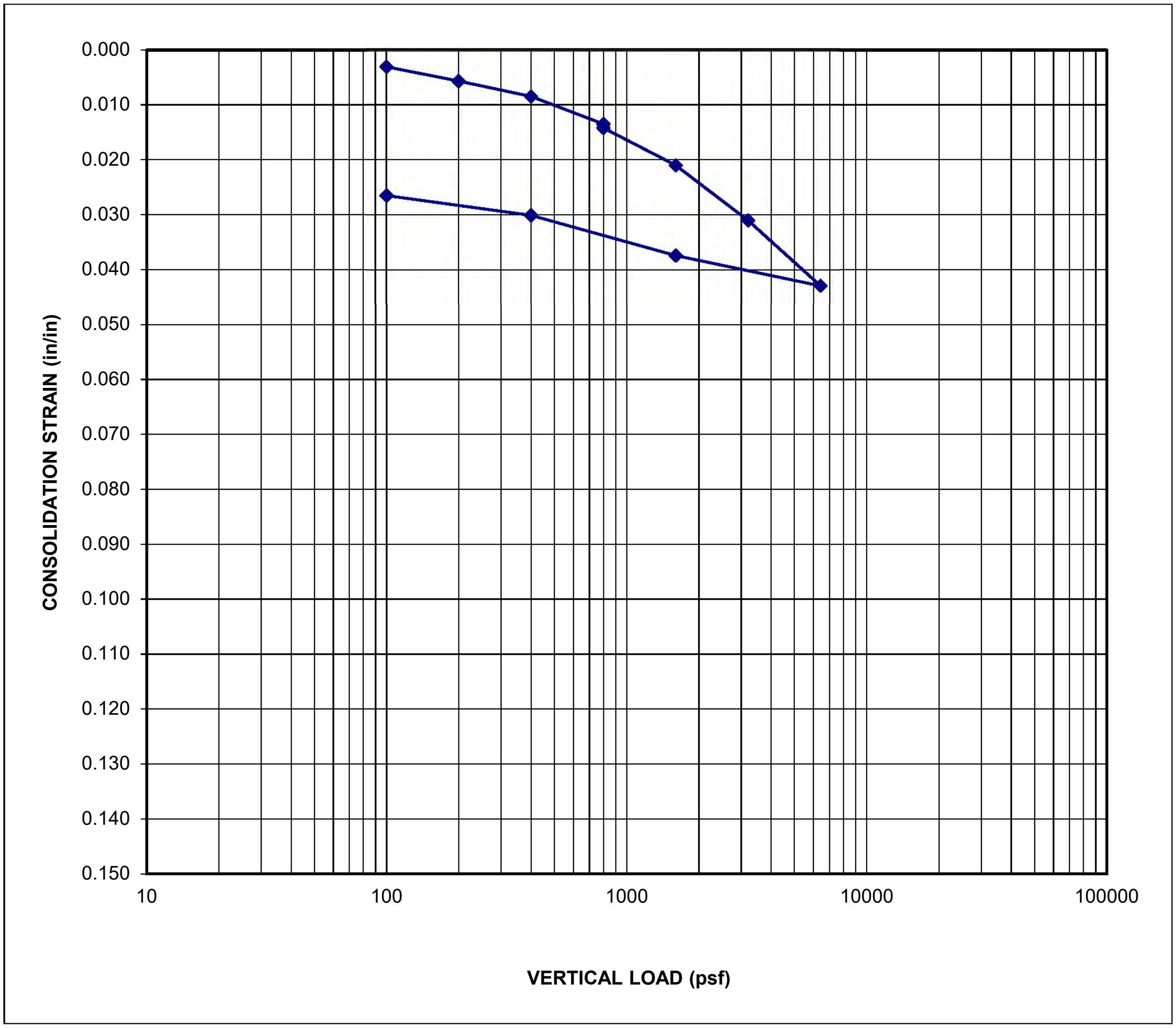
Project No.: 2G-2009003

Figure No.: **6**

GILES ENGINEERING ASSOCIATES, INC.

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

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(ML) - Sandy Silt with Clay		
Boring No.	**B-20**		
Sample No.	4-CS	Initial Moisture Content (%)	37.0
Depth (ft.)	**10 ft**	Final Moisture Content (%)	40.2
Elevation (ft.)	83	Natural Density (pcf)	97.9
Liquid Limit	N/A	Initial Dry Density (pcf)	71.5
Plastic Limit	N/A	Final Dry Density (pcf)	71.6
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.07%
Initial Specimen Thickness (in.)	1.00		

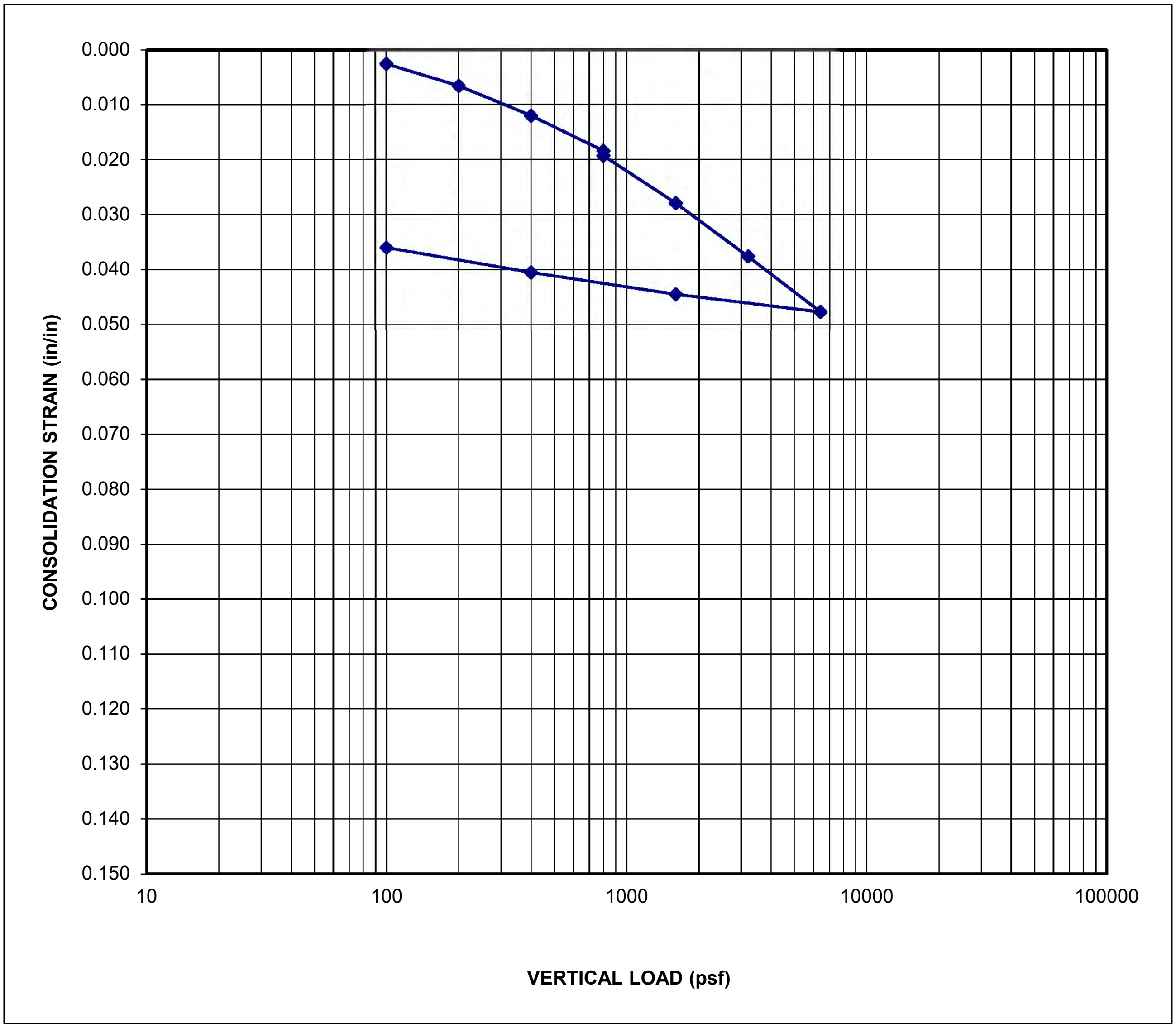
Sample inundated with water at 800 psf pressure

Project:	Public Storage Los Angeles
Client:	Public Storage
Project No.:	2G-2009003
Figure No.:	**7**

GILES ENGINEERING ASSOCIATES, INC.

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

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	(SP) - Poorly-graded fine Sand		
Boring No.	**B-20**		
Sample No.	6-CS	Initial Moisture Content (%)	7.0
Depth (ft.)	**20 ft**	Final Moisture Content (%)	19.4
Elevation (ft.)	73	Natural Density (pcf)	109.4
Liquid Limit	N/A	Initial Dry Density (pcf)	103.1
Plastic Limit	N/A	Final Dry Density (pcf)	103.2
Specimen Diameter (in.)	2.42	Collapse at 800 psf (%)	0.09%
Initial Specimen Thickness (in.)	1.00		

Sample inundated with water at 800 psf pressure

Project:	Public Storage Los Angeles
Client:	Public Storage
Project No.:	2G-2009003
Figure No.:	**8**

GILES ENGINEERING ASSOCIATES, INC.

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

BORING NO. & LOCATION:
B- 1

SURFACE ELEVATION:
96 feet

COMPLETION DATE:
10/09/20

FIELD REP:
ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 3.5 inches of asphaltic concrete			1-SS	7				12		
Dark Brown Silty fine Sand with Gravel - Moist (Certified Fill)			2-SS	28				12		
Light Brown Silty fine Sand- Damp (Native)		90	3-SS	5				4		
Dark Olive Brown- Very Moist	10		4-SS	7				19		
Light Yellow Brown- Moist		80	5-SS	20				12		P_{200}=35%
Light Yellow Brown, medium to coarse grained, little Silt - Moist	20		6-SS	41				6		P_{200}=14%
Trace Gravel - Damp		70	7-SS	88				3		
Fine grained Sand - Damp	30		8-SS	55				4		
Light Greenish Gray to Brown fine Sand		60	9-SS	86				2		
	40		10-SS	50/4"				3		
Mottled Yellow Brown to Red fine to medium grained		50	11-SS	46				4		
Gray, Silt - Very Moist	50		12-SS	14				23		LL=38 PL=33 PI=5
		40	13-SS	9				29		
Gray Silty fine Sand - Very Moist	60		14-SS	52				11		

Boring Terminated at about 61.5 feet (EL. 34.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	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- 2/P-1

SURFACE ELEVATION: 98 feet

COMPLETION DATE: 10/09/12

FIELD REP: ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphaltic concrete			1-SS	7				8		
Dark Brown Silty fine Sand - Moist (Certified Fill)			2-SS	7				8		
Red Brown fine to medium Sand - Moist (Certified Fill)			3-CS	15				8		Dd=96.2 pcf
		90								
Dark Brown Silty fine Sand - Moist (Certified Fill)	10		4-CS	33				12		Dd=105.4 pcf
Dark Brown Silty fine Sand, trace Gravel - Moist (Native)										
Dark Brown to Black, some organics - Very Moist			5-SS	6				20		LOI=20.0%
		80								
Dark Gray to Black fine Sandy Clay, trace organics - Very Moist	20		6-SS	6				35		LOI=9.2%
Yellow Brown to Gray medium to coarse Sand, trace Gravel - Damp			7-SS	15				6		P_{200}=11%
		70								
Yellow Brown to Light Brown fine to medium grained	30		8-SS	65				4		
Light Gray fine to medium grained			9-SS	73				3		
Boring Terminated at about 36.5 feet (EL. 61.5')										

Water Observation Data

▽ Water Encountered During Drilling: None
▼ Water Level At End of Drilling:
▨ Cave Depth At End of Drilling:
▼ Water Level After __ Hours: __ ft.
▬ Cave Depth After __ Hours: __ ft.

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

LOI = Loss on Ignition

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: 98 feet	PUBLIC STORAGE FACILITY 5741 WEST JEFFERSON BOULEVARD LOS ANGELES, CA PROJECT NO: 2G-2009003	
COMPLETION DATE: 10/09/20		
FIELD REP: ROBERT TORRES		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphaltic concrete			1-SS	21				6		
Dark Brown Silty fine Sand, trace Gravel - Moist (Certified Fill)			2-SS	10				9		
Trace Clay with Asphalt			3-CS	11				12		Dd=122.8 pcf
		90								
More Asphalt - Very Moist	10		4-CS	4				14		
Dark Brown Silty Clay, trace Sand - Very Moist (Native)			5-SS	12				30		
		80								
	20									
Light Brown Silty fine Sand - Moist			6-SS	33				8		
Some Gravel - Damp			7-SS	67				3		
		70								
	30		8-SS	97				5		
			9-SS	68				3		

Boring Terminated at about 36.5 feet (EL. 61.5')

Water Observation Data

- ▽ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▒ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

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

SURFACE ELEVATION:
97 feet

COMPLETION DATE:
10/09/20

FIELD REP:
ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphaltic concrete			1-SS	4				8		
Dark Brown Silty fine Sand with Gravel - Damp (Certified Fill)			2-SS	16				15		
Dark Brown Silty fine Sand, trace Clay - Very Moist (Native)		90	3-CS	38				30		
Dark Brown to Light Brown Clayey fine Sand, trace Silt - Very Moist	10		4-SS	7				12		
Dark Brown Silty fine Sand, trace Clay - Very Moist										
			5-SS	15				17		
		80								
White medium to coarse Sand with Gravel - Damp	20		6-SS	50/6"				2		
White to Light Brown Silty fine Sand with Gravel - Damp			7-SS	54				7		
		70								
Olive Brown- Very Moist	30		8-SS	47				14		
Damp			9-SS	48				5		

Boring Terminated at about 36.5 feet (EL. 60.5')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

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.

TEST BORING LOG

BORING NO. & LOCATION:
B- 5

SURFACE ELEVATION:
98 feet

COMPLETION DATE:
10/09/20

FIELD REP:
ROBERT TORRES

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete										
Dark Brown Silty fine Sand, trace Clay and Gravel - Very Moist to Damp (Certified Fill)			1-SS	9				19		
			2-SS	9				12		
			3-SS	8				2		
		90								
Brown Sandy Gravel - Dry (Certified Fill)	10		4-SS	8				2		
Black Silty fine Sand, trace Clay and Gravel, little organics - Very Moist (Native)			5-SS	5				19		LOI=17.8%
		80								
Light Brown - Damp	20		6-SS	33				5		
			7-SS	55				3		
		70								
Trace Gravel - Damp	30		8-SS	50				3		
More Gravel			9-SS	50/6"				5		
		60								
No Recovery	40		10-SS	69						

Boring Terminated at about 41.5 feet (EL. 56.5')

Water Observation Data

- ∇ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▨ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test

LOI = Loss on Ignition

BORING NO. & LOCATION:
B- 6/P-2

SURFACE ELEVATION:
98 feet

COMPLETION DATE:
10/09/20

FIELD REP:
ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete			1-SS	19				9		
Dark Brown Silty fine Sand with Gravel - Moist (Certified Fill)			2-SS	11				7		
Dark Brown to Light Brown Silty fine Sand - Moist (Native)			3-SS	8				7		
		90								
Dark Brown to Olive	10		4-SS	9				11		P_{200}=36%
Light Brown fine to coarse grained - Damp			5-SS	22				5		P_{200}=16%
		80								
Light Brown Silty fine Sand, trace Clay - Damp	20		6-SS	13				2		P_{200}=45%
			7-SS	31				3		
		70								
Light Brown fine Sand - Moist	30		8-SS	62				9		
Light Brown to Olive Silty fine Sand - Moist			9-SS	67				6		P_{200}=18%
		60								
Olive - Damp	40		10-SS	67				4		P_{200}=24%
			11-SS	63				8		
		50								
	50		12-SS	84				5		
			13-SS	87				5		
		40								
	60		14-SS	93				5		

Boring Terminated at about 61.5 feet (EL. 36.5')

	Water Observation Data	Remarks:
⊻	Water Encountered During Drilling: None	SS = Standard Penetration Test
⊻	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

SURFACE ELEVATION:
96 feet

COMPLETION DATE:
05/20/21

FIELD REP:
ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 4 inches of aggregate base		95								
Dark Brown Silty Sand, trace Clay - Moist (Certified Fill)			1-SS	10				10		
Dark Brown to Brown Silty Clay, trace Sand - Moist (Native)	5		2-SS	10				13		
Dark Brown to Brown Silty Sand, trace Clay - Moist		90	3-SS	7				37		
Brown Silty Clay - Moist	10	85	4-SS	10				17		
Brown Silty fine Sand - Moist	15	80	5-SS	23				11		
	20	75	6-SS	41				4		

Boring Terminated at about 21.5 feet (EL. 74.5')

Water Observation Data

∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
■	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test

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- 8

SURFACE ELEVATION: 98 feet

COMPLETION DATE: 05/20/21

FIELD REP: ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 5 inches of aggregate base										
Brown Silty Sand - Moist (Certified Fill)			1-SS	8				12		
		95								
			2-SS	8				9		
	5									
Brown Silty Clay, trace Sand - Moist (Certified Fill)			3-SS	5				24		
		90								
	10									
Dark Brown, trace Gravel, trace asphalt, brick, and wood fibers - Moist			4-SS	6				19		
Black to Brown Silty Sand, organics - Very Moist (Native)										
		85								
	15									
			5-SS	6				17		
		80								
	20									
Gray, fine grained - Moist			6-SS	16				10		

Boring Terminated at about 21.5 feet (EL. 76.5')

Water Observation Data

- ▽ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▒ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test

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
SURFACE ELEVATION:	96 feet
COMPLETION DATE:	05/20/21
FIELD REP:	ROBERT TORRES

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 2 inches of asphaltic concrete over 6 inches of aggregate base		95								
Dark Brown Silty Clay with Gravel - Moist (Certified Fill)			1-SS	13				5		
Dark Brown Silty Clay - Moist (Native)			2-SS	12				1		
	5	90	3-SS	6				3		
Dark Brown to Brown some Gravel - Moist	10	85	4-SS	8				5		
Dark Brown Silty Sand, trace Clay - Moist	15	80	5-SS	9				9		
Dark Brown Silty Clay, trace Sand, organics - Moist	20	75	6-SS	8				6		

Boring Terminated at about 21.5 feet (EL. 74.5')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	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: 98 feet	PUBLIC STORAGE FACILITY 5741 WEST JEFFERSON BOULEVARD LOS ANGELES, CA PROJECT NO: 2G-2009003	
COMPLETION DATE: 05/20/21		
FIELD REP: ROBERT TORRES		

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 2 inches of asphaltic concrete over 6 inches of aggregate base										
Dark Brown Silty Clay, trace Gravel and Sand - Moist (Certified Fill)			1-SS	12				3		
		95								
			2-SS	5				10		
	5									
Dark Brown Silty Clay, trace Sand and Gravel - Moist (Native)			3-SS	5				10		
		90								
	10									
			4-SS	10				11		
		85								
	15									
Dark Brown to Black, organics, trace Sand - Moist			5-SS	10				9		
		80								
	20									
Brown coarse Sand - Damp			6-SS	26				4		

Boring Terminated at about 21.5 feet (EL. 76.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	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.

TEST BORING LOG

BORING NO. & LOCATION:
B-11

SURFACE ELEVATION:
97 feet

COMPLETION DATE:
05/20/21

FIELD REP:
ROBERT TORRES

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-2009003

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 6 inches of aggregate base										
Brown Silty fine Sand with Gravel - Moist (Certified Fill)		95	1-SS	8				7		
Dark Brown, trace Clay - Damp	5		2-SS	15				4		
Dark Brown Silty fine Sand, trace Clay - Moist (Native)		90	3-SS	6				8		
Brown	10	85	4-SS	11				11		
	15	80	5-SS	22				9		
	20		6-SS	43				5		

Boring Terminated at about 21.5 feet (EL. 75.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	SS = Standard Penetration Test
▼	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-12	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 93 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 01/04/24	5741 WEST JEFFERSON BOULEVARD LOS ANGELES, CALIFORNIA	
FIELD REP: R. RAMOS & J. GONZALEZ	PROJECT NO: 2G-2009003-R	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 1-inch-thick Asphaltic Concrete over 5-inch-thick Aggregate Base										
Dark Brown, Clayey Sand, fine to medium grained - Moist (Certified Fill)			1-SS	12				17		
		90								
Some Gravel			2-CS	29				18		Dd = 111.4 pcf %P(1) = 92
	5									
Brown, Silty Sand, fine grained - Very Moist (Native)			3-SS	5				40		
		85								
	10									
Olive Brown - Moist			4-CS	36				14		Dd = 115.2 pcf
		80								
	15									
Some Clay			5-SS	12	2.4			18		
		75								
	20									
Yellow, fine to medium Sand, some fine Gravel - Moist			6-CS	68				6		Dd = 108.0 pcf

Boring Terminated at about 21.5 feet (EL. 71.5')

	Water Observation Data
∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▒	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(1) = Percent Dry Density in Regards to Proctor #1 of 121.0 @ 12.5%

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-13

SURFACE ELEVATION:
93 feet

COMPLETION DATE:
01/04/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 1-inch-thick Asphaltic Concrete over 6-inch-thick Aggregate Base										
Brown, Clayey Sand, medium to coarse Sand, trace asphalt debris - Moist (Certified Fill)		90	1-CS	11		2.0		17		Dd = 110.5 pcf %P (1)= 91
Brown, Sandy Clay, fine to medium Sand - Moist (Native)	5		2-SS	2		0.75		21		
Brown, Silty Sand, coarse grained, trace Clay - Moist		85	3-CS	25				22		
Olive Brown, Sandy Clay - Moist	10	80	4-SS	8	1.9	2.25		22		Dd = 101.5 pcf
Yellowish Brown, Sand, trace Silt - Moist	15	75	5-CS	33				7		Dd = 111.9 pcf
Yellow, trace Gravel- Damp	20		5-SS	39				4		

Boring Terminated at about 21 feet (EL. 72')

	Water Observation Data
∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
■	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(1) = Percent Dry Density in Regards to Proctor #1 of 121.0 @ 12.5%

BORING NO. & LOCATION:
B-14

SURFACE ELEVATION:
92 feet

COMPLETION DATE:
01/04/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 1-inch-thick Asphaltic Concrete over 4-inch-thick Aggregate Base										
Brown, Sandy Clay, trace Silt - Moist (Certified Fill)		90	1-SS	4	1.3	1.8		18		
Brown, Sandy Silt with Clay, abundant Gravel fragments - Moist (Certified Fill)	5		2-CS	28		3.25		15		Dd = 103.3 pcf %P(3) = 99
Brown, Silt, trace fine Sand - Moist (Certified Fill)		85	3-SS	7	1.8	1.25		18		
Brown, Silt, trace Clay, little Organics - Moist (Native)	10	80	4-CS	15		1.25		38		Dd = 74.6 pcf LOI = 11%
Olive Brown, Clay, trace fine Sand - Moist	15	75	5-SS	10	2.0	2.0		25		
Yellowish Brown, 3 inches Clay with Sand, 3 inches Sand with Gravel - Moist	20		6-CS	50/5.5"				5		Dd = 110.7 pcf

Boring Terminated at about 21 feet (EL. 71')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(3) = Percent Dry Density in Regards to Proctor #3 of 104.5 @ 19%

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-15

SURFACE ELEVATION:
92 feet

COMPLETION DATE:
01/03/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3-inch-thick Asphaltic Concrete over 4-inch-thick Aggregate Base										
Dark Brown, Sandy Clay, some Gravel - Moist (Certified Fill)		90	1-SS	12		1.25		16		
			2-CS	7		2.75		13		Dd = 102.0 pcf %P(3) = 98
	5									
Brown, Silty Clay, little Organics - Moist (Native)		85	3-SS	4				76		LOI = 17%
some Gravel	10		4-CS	14		2.5		32		Dd = 67.8 pcf LOI = 14%
		80								
Dark Brown, Sandy Silt, fine Sand - Moist	15		5-SS	15				14		
Yellowish Brown, poorly graded Sand, fine to medium grained, some Silt -Moist		75								
Brown, Sandy Silt, fine Sand - Moist	20		6-CS	50/5"				5		Dd = 108.8 pcf
Yellowish Brown, poorly graded Sand, fine to medium Sand, some fine Gravel - Moist										
Boring Terminated at about 21 feet (EL. 71')										

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(3) = Percent Dry Density in Regards to Proctor #3 of 104.5 @ 19%

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-16

SURFACE ELEVATION:
93 feet

COMPLETION DATE:
01/03/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3-inch-thick Asphaltic Concrete over 3-inch-thick Aggregate Base										
Brown Sandy Silt, fine Sand, trace Clay - Moist (Certified Fill)			1-SS	8		1.0		8		
Brown, Silty Sand, fine to medium grained - Moist (Certified Fill)	5	90	2-CS	5				14		Dd = 98.6 pcf %P(3) = 94
Redish Brown, Sandy Clay, fine to medium sand, trace Gravel - Moist (Certified Fill)			3-SS	9	1.0			20		
Dark Brown, Clayey Sand, fine grained, trace Gravel, trace Brick Fragments (Certified Fill)	10	85	4-CS	10				13		Dd = 98.2 pcf
Dark Brown, Clayey Silt, trace fine to medium Sand - Moist (Certified Fill) Wood and Stone fragments		80	5-SS	8		1.5		44		LOI = 22%
Gray to Dark Gray, Sandy Silt, fine Sand, little Organics - Very Moist (Native) Dark Gray, trace Organics - Very Moist	15		6-CS	9		2.0		39		Dd = 74.9 pcf LOI = 9%
Brown, poorly graded Sand, fine Sand, some Clay - Moist		75	7-SS	7	1.7	1.5		28		
Olive Gray, Clayey Sand, fine grained, some medium grained - Moist	20	70	8-CS	15		0.5		21		Dd = 90.8 pcf
Gray, poorly graded Sand, fine to medium grained- Moist	25		9-SS	21				21		
Olive Gray, Sandy Silt, fine Sand, some Clay - Moist	30	65	10-CS	39		> 4.5		24		Dd = 98.6 pcf

Boring Terminated at about 31.5 feet (EL. 61.5')

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▒	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
■	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(3) = Percent Dry Density in Regards to Proctor #3 of 104.5 @ 19%

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-17

TEST BORING LOG

SURFACE ELEVATION:
93 feet

PUBLIC STORAGE FACILITY

COMPLETION DATE:
01/03/24

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

FIELD REP:
R. RAMOS & J. GONZALEZ

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3-inch-thick Asphaltic Concrete over 4-inch-thick Aggregate Base										
Dark Gray, Sandy Silt, fine Sand, trace Clay, some Gravel - Moist (Certified Fill)		90	1-CS	14		1.5		23		Dd = 97.7 pcf %P(3) = 93
Light Brown, Silty Sand, fine grained, some Gravel - Moist (Native)	5		2-SS	8				8		
Brown, Silty Clay - Very Moist			3-CS	15		1.5		40		Dd = 80.0 pcf
Olive Brown, Sandy Silt, fine Sand, trace Clay - Very Moist	10	85	4-SS	10	2.0	2.5		20		
		80								
Moist	15		5-CS	33		3.25		16		Dd = 110.4 pcf
Yellowish Brown, Sand, medium to coarse grained - Damp		75	6-SS	52				2		
some Gravel	20		7-CS	50/6"						

Boring Terminated at about 20.5 feet (EL. 72.5')

Water Observation Data

- ▽ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▨ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(3) = Percent Dry Density in Regards to Proctor #3 of 104.5 @ 19%

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-18

SURFACE ELEVATION:
94 feet

COMPLETION DATE:
01/03/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 1-inch-thick Asphaltic Concrete over 3-inch-thick Aggregate Base										
Dark Brown, Silty Sand, some Clay - Moist (Certified Fill)			1-CS	18				14		Dd = 109.8 pcf %P(2) = 92
Dark Brown, Sandy Clay, fine to medium Sand - Moist (Native)		90	2-SS	13	2.2	2.0		27		
	5									
Light Brown, trace Gravel, little Organics - Very Moist			3-CS	23		4.0		53		Dd = 65.4 pcf LOI = 14%
		85								
	10									
Olive Brown, Silty Sand, fine grained - Moist			4-SS	14				12		
		80								
	15									
Brown, fine to medium grained			5-CS	41				10		Dd = 108.7 pcf
		75								
	20									
Olive Brown			6-SS	17				14		
		70								
	25									
Dark Gray, Clayey Sand, medium to coarse grained - Moist			7-CS	50/6"				12		Dd = 121.3 pcf

Boring Terminated at about 26.5 feet (EL. 67.5')

	Water Observation Data
∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(2) = Percent Dry Density in Regards to Proctor #2 of 119.5 @ 12.0%

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-19

SURFACE ELEVATION:
93 feet

COMPLETION DATE:
01/04/24

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2-inch-thick Asphaltic Concrete over 6-inch-thick Aggregate Base										
Brown, Clayey Sand, some Gravel - Moist (Certified Fill)			1-CS	22				23		Dd = 110.0 pcf %P(1) = 91
		90								
Asphalt debris			2-SS	8				16		
	5									
Trace Silt			3-CS	18				18		Dd = 109.5 pcf %P 91
		85								
Dark Gray, Sandy Silt, fine to medium Sand, little to some Organics - Very Moist (Native)	10		4-SS	6		0.5		107		LOI = 16%
		80								
	15		5-CS	11		3.0		63		Dd = 73.2 pcf LOI = 32%
Yellowish Brown, poorly graded Sand, fine to medium grained, some Gravel - Very Moist		75								
Moist	20		7-SS	27				7		

Boring Terminated at about 21.5 feet (EL. 71.5')

Water Observation Data

- ▽ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▨ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(1) = Percent Dry Density in Regards to Proctor #1 of 121.0 @ 12.5%

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.

TEST BORING LOG

BORING NO. & LOCATION: B-20

SURFACE ELEVATION: 93 feet

COMPLETION DATE: 01/04/24

FIELD REP: R. RAMOS & J. GONZALEZ

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2-inch-thick Asphaltic Concrete over 6-inch-thick Aggregate Base										
Grayish Brown, Sandy Clay, fine to medium Sand - Moist (Certified Fill)		90	1-SS	7				23		
Brown, Silty Sand, fine to medium grained, some Clay - Moist (Certified Fill)	5		2-CS	18				17		Dd = 111.6 pcf %P(2) = 93
Brown, Sandy Clay - Moist (Native)		85	3-SS	8	2.4	2.75		27		
Light to Dark Brown, Sandy Silt with Clay - Moist	10	80	4-CS	8		3.5		37		Dd = 73.8 pcf
Olive Brown, Silty Sand - Moist	15	75	5-SS	15				12		
Yellowish Brown, poorly graded Sand, fine grained - Moist	20		6-CS	50/4"				7		Dd = 103.1 pcf

Boring Terminated at about 21 feet (EL. 72')

	Water Observation Data
∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▒	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
▬	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(2) = Percent Dry Density in Regards to Proctor #2 of 119.5 @ 12.0%

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-21

SURFACE ELEVATION:
92 feet

COMPLETION DATE:
01/04/23

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 1-inch-thick Asphaltic Concrete over 5-inch-thick Aggregate Base										
Brown, Silty Sand with Clay, fine to medium Sand - Moist (Certified Fill)		90	1-CS	12		2.0		13		Dd = 109.9 pcf %P(2) = 92
Brown, Silty Sand, coarse grained, some Gravel - Moist (Native)	5		2-SS	5				12		
		85	3-CS	18				15		Dd = 136.8 pcf
Yellowish Brown, Clayey Sand, coarse grained - Moist	10		4-SS	31		1.25		22		
Dark Brown, Sandy Clay, some Gravel - Moist		80								
Dark Brown, Silt, trace Clay, trace Organics - Very Moist	15		5-CS	9		1.25		64		Dd = 58.6 pcf LOI = 9%
		75								
	20		6-SS	8	0.5	0.5		57		LOI = 9%

Boring Terminated at about 21.5 feet (EL. 70.5')

Water Observation Data

∇	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
▨	Cave Depth At End of Drilling:
▼	Water Level After __ Hours: __ ft.
■	Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(2) = Percent Dry Density in Regards to Proctor #2 of 119.5 @ 12.0%

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-22

SURFACE ELEVATION:
95 feet

COMPLETION DATE:
01/04/23

FIELD REP:
R. RAMOS & J. GONZALEZ

TEST BORING LOG

PUBLIC STORAGE FACILITY

5741 WEST JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA

PROJECT NO: 2G-2009003-R

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2-inch-thick Asphaltic Concrete over 6-inch-thick Aggregate Base										
Brown, Silty Sand, trace Clay - Moist (Certified Fill)			1-SS	4				17		
Brown, Sandy Clay, fine to medium Sand, trace Gravel - Moist (Certified Fill)	5	90	2-CS	14		3.25		17		Dd =98.5 pcf %P(3) = 94
Light Gray to Dark Brown, Silty Sand, fine to medium grained - Moist (Certified Fill)			3-SS	4				8		
Gray, Sandy Clay, fine Sand - Moist (Native)	10	85	4-CS	26		4.0		14		Dd = 114.5 pcf %P = 94
Light Brown, Silty Sand, fine grained - Moist	15	80	5-SS	13				20		
	20	75	6-CS	50/6"				9		Dd = 110.3 pcf
Yellowish Brown, poorly graded Sand, fine to medium grained - Moist										
Boring Terminated at about 21 feet (EL. 74')										

Water Observation Data

- ∇ Water Encountered During Drilling: None
- ▼ Water Level At End of Drilling:
- ▨ Cave Depth At End of Drilling:
- ▼ Water Level After __ Hours: __ ft.
- ▬ Cave Depth After __ Hours: __ ft.

Remarks:

SS = Standard Penetration Test
CS = California Split Spoon
Elevations Based on Google Earth
%P(3) = Percent Dry Density in Regards to Proctor #3 of 104.5 @ 19.0%

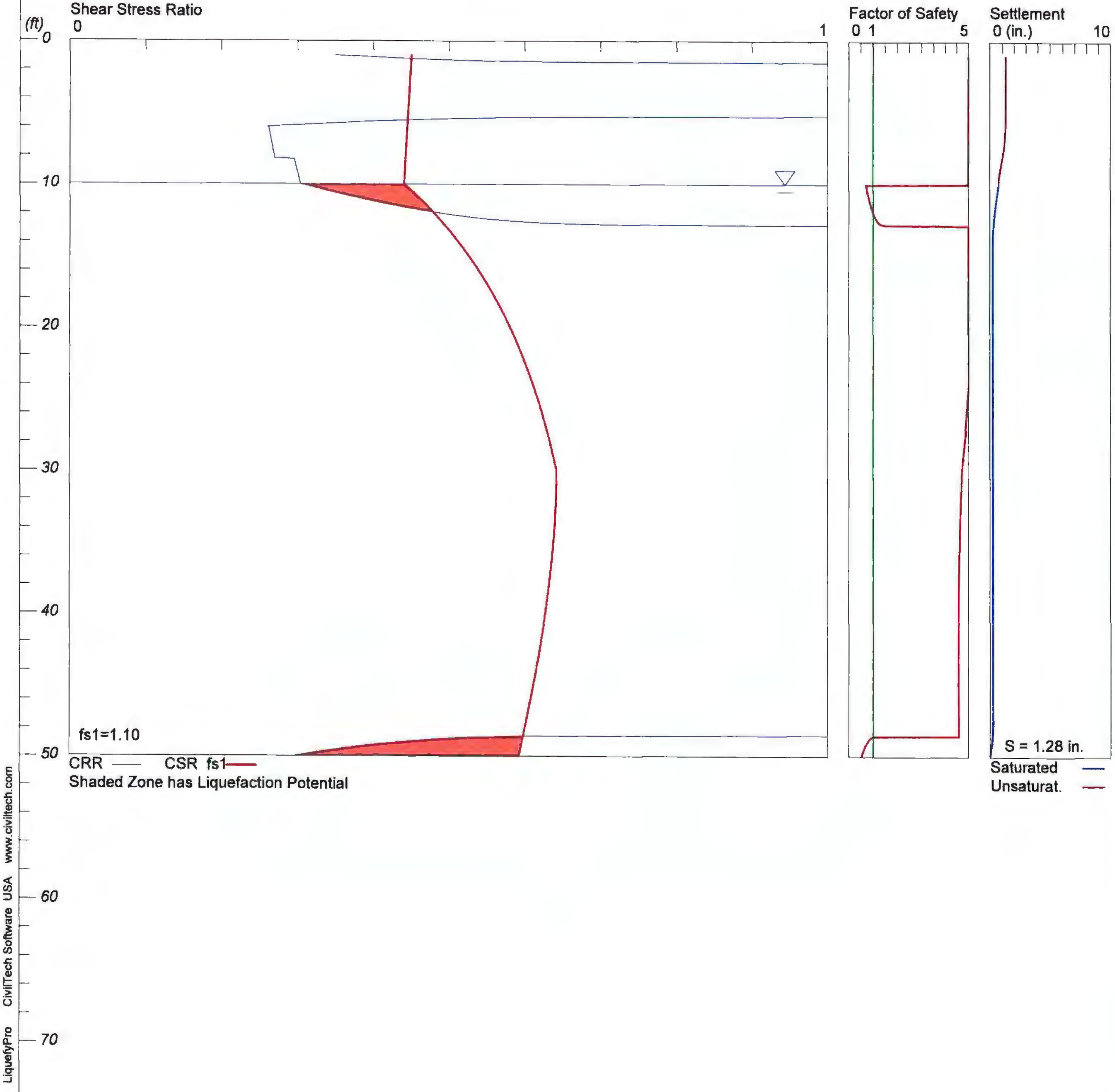
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

Boring B-1 - 2/3 PGA ; 475 yr Rtrn; FS 1.1

Hole No.=B-1 ***Water Depth=10 ft*** ***Surface Elev.=96 ft***

Magnitude=6.35
Acceleration=0.63g



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**********************************************************************************
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                                          LIQUEFACTION ANALYSIS SUMMARY

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   2/20/2024       4:12:00 PM

        Input File Name: C:\Users\jmaier\Desktop\Culver City CBC 2022 Liq 2-3 PGA -
475 Rtrn - FS 1.1.liq
        Title:  Boring B-1 - 2/3 PGA ; 475 yr Rtrn; FS 1.1
        Subtitle:  PS Los Angeles CA – CBC 2022

        Surface Elev.=96 ft
        Hole No.=B-1
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration= 0.63 g
        Earthquake Magnitude= 6.35

 Input Data:
        Surface Elev.=96 ft
        Hole No.=B-1
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration=0.63 g
        Earthquake Magnitude=6.35
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu/Seed
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                  Ce = 1.25
        7. Borehole Diameter,                                        Cb= 1
        8. Sampling Method,                                         Cs= 1.2
        9. User request factor of safety (apply to CSR) ,   User= 1.1
           Plot one CSR curve (fs1=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options
```

```
        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        1.00    7.00    120.00  70.00
        3.50    28.00   120.00  30.00
        6.00    5.00    120.00  30.00
        10.00   7.00    120.00  30.00
        15.00   20.00   120.00  35.00
        20.00   41.00   120.00  14.00
        25.00   88.00   120.00  10.00
        30.00   55.00   120.00  10.00
        35.00   86.00   120.00  10.00
        40.00   100.00  120.00  10.00
        45.00   46.00   120.00  10.00
        50.00   14.00   120.00  NoLiq
        ____________________________________

Output Results:
        Settlement of Saturated Sands=0.71 in.
        Settlement of Unsaturated Sands=0.57 in.
        Total Settlement of Saturated and Unsaturated Sands=1.28 in.
        Differential Settlement=0.638 to 0.842 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        1.00    0.35    0.45    5.00    0.71    0.57    1.28
        1.50    3.06    0.45    5.00    0.71    0.57    1.27
        2.00    3.06    0.45    5.00    0.71    0.57    1.27
        2.50    3.06    0.45    5.00    0.71    0.57    1.27
        3.00    3.06    0.45    5.00    0.71    0.57    1.27
        3.50    3.06    0.45    5.00    0.71    0.57    1.27
        4.00    3.06    0.45    5.00    0.71    0.57    1.27
        4.50    3.06    0.45    5.00    0.71    0.57    1.27
        5.00    3.06    0.45    5.00    0.71    0.56    1.27
        5.50    0.46    0.44    5.00    0.71    0.56    1.27
        6.00    0.26    0.44    5.00    0.71    0.55    1.26
        6.50    0.26    0.44    5.00    0.71    0.53    1.23
        7.00    0.27    0.44    5.00    0.71    0.48    1.19
        7.50    0.27    0.44    5.00    0.71    0.41    1.12
        8.00    0.27    0.44    5.00    0.71    0.31    1.02
        8.50    0.30    0.44    5.00    0.71    0.21    0.91
        9.00    0.30    0.44    5.00    0.71    0.11    0.82
        9.50    0.30    0.44    5.00    0.71    0.01    0.72
        10.00   0.30    0.44    0.69*   0.71    0.00    0.71
        10.50   0.34    0.45    0.76*   0.61    0.00    0.61
        11.00   0.38    0.46    0.83*   0.53    0.00    0.53
        11.50   0.43    0.47    0.91*   0.45    0.00    0.45
        12.00   0.49    0.48    1.01    0.39    0.00    0.39
```

12.50	0.57	0.49	1.16	0.33	0.00	0.33
13.00	3.06	0.50	5.00	0.28	0.00	0.28
13.50	3.06	0.50	5.00	0.25	0.00	0.25
14.00	3.06	0.51	5.00	0.23	0.00	0.23
14.50	3.06	0.52	5.00	0.22	0.00	0.22
15.00	3.06	0.53	5.00	0.22	0.00	0.22
15.50	3.06	0.53	5.00	0.22	0.00	0.22
16.00	3.06	0.54	5.00	0.22	0.00	0.22
16.50	3.06	0.54	5.00	0.22	0.00	0.22
17.00	3.06	0.55	5.00	0.22	0.00	0.22
17.50	3.06	0.56	5.00	0.22	0.00	0.22
18.00	3.06	0.56	5.00	0.22	0.00	0.22
18.50	3.06	0.57	5.00	0.22	0.00	0.22
19.00	3.06	0.57	5.00	0.22	0.00	0.22
19.50	3.06	0.58	5.00	0.22	0.00	0.22
20.00	3.06	0.58	5.00	0.22	0.00	0.22
20.50	3.06	0.58	5.00	0.22	0.00	0.22
21.00	3.06	0.59	5.00	0.22	0.00	0.22
21.50	3.06	0.59	5.00	0.22	0.00	0.22
22.00	3.06	0.60	5.00	0.22	0.00	0.22
22.50	3.06	0.60	5.00	0.22	0.00	0.22
23.00	3.06	0.60	5.00	0.22	0.00	0.22
23.50	3.06	0.61	5.00	0.22	0.00	0.22
24.00	3.06	0.61	5.00	0.22	0.00	0.22
24.50	3.06	0.61	4.99	0.22	0.00	0.22
25.00	3.06	0.62	4.97	0.22	0.00	0.22
25.50	3.06	0.62	4.94	0.22	0.00	0.22
26.00	3.06	0.62	4.92	0.22	0.00	0.22
26.50	3.06	0.62	4.90	0.22	0.00	0.22
27.00	3.06	0.63	4.88	0.22	0.00	0.22
27.50	3.07	0.63	4.88	0.22	0.00	0.22
28.00	3.06	0.63	4.85	0.22	0.00	0.22
28.50	3.06	0.63	4.81	0.22	0.00	0.22
29.00	3.05	0.64	4.78	0.22	0.00	0.22
29.50	3.04	0.64	4.75	0.22	0.00	0.22
30.00	3.03	0.64	4.72	0.22	0.00	0.22
30.50	3.02	0.64	4.71	0.22	0.00	0.22
31.00	3.01	0.64	4.70	0.22	0.00	0.22
31.50	3.00	0.64	4.69	0.22	0.00	0.22
32.00	3.00	0.64	4.68	0.22	0.00	0.22
32.50	2.99	0.64	4.67	0.22	0.00	0.22
33.00	2.98	0.64	4.66	0.22	0.00	0.22
33.50	2.97	0.64	4.65	0.22	0.00	0.22
34.00	2.96	0.64	4.64	0.22	0.00	0.22
34.50	2.95	0.64	4.63	0.22	0.00	0.22
35.00	2.95	0.64	4.62	0.22	0.00	0.22
35.50	2.94	0.64	4.62	0.22	0.00	0.22
36.00	2.93	0.64	4.61	0.22	0.00	0.22
36.50	2.92	0.63	4.60	0.22	0.00	0.22
37.00	2.91	0.63	4.60	0.22	0.00	0.22

```
        37.50   2.91    0.63    4.59    0.22    0.00    0.22
        38.00   2.90    0.63    4.59    0.22    0.00    0.22
        38.50   2.89    0.63    4.58    0.22    0.00    0.22
        39.00   2.88    0.63    4.58    0.22    0.00    0.22
        39.50   2.87    0.63    4.58    0.22    0.00    0.22
        40.00   2.87    0.63    4.57    0.22    0.00    0.22
        40.50   2.86    0.63    4.57    0.22    0.00    0.22
        41.00   2.85    0.62    4.57    0.22    0.00    0.22
        41.50   2.84    0.62    4.57    0.22    0.00    0.22
        42.00   2.84    0.62    4.57    0.22    0.00    0.22
        42.50   2.83    0.62    4.57    0.22    0.00    0.22
        43.00   2.82    0.62    4.57    0.22    0.00    0.22
        43.50   2.81    0.62    4.57    0.22    0.00    0.22
        44.00   2.81    0.61    4.57    0.22    0.00    0.22
        44.50   2.80    0.61    4.57    0.22    0.00    0.22
        45.00   2.79    0.61    4.57    0.22    0.00    0.22
        45.50   2.79    0.61    4.57    0.22    0.00    0.22
        46.00   2.78    0.61    4.57    0.22    0.00    0.22
        46.50   2.77    0.61    4.58    0.22    0.00    0.22
        47.00   2.76    0.60    4.58    0.22    0.00    0.22
        47.50   2.76    0.60    4.58    0.22    0.00    0.22
        48.00   2.75    0.60    4.59    0.22    0.00    0.22
        48.50   2.74    0.60    4.59    0.20    0.00    0.20
        49.00   0.44    0.60    0.74*   0.14    0.00    0.14
        49.50   0.35    0.59    0.60*   0.07    0.00    0.07
        50.00   0.29    0.59    0.50*   0.00    0.00    0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

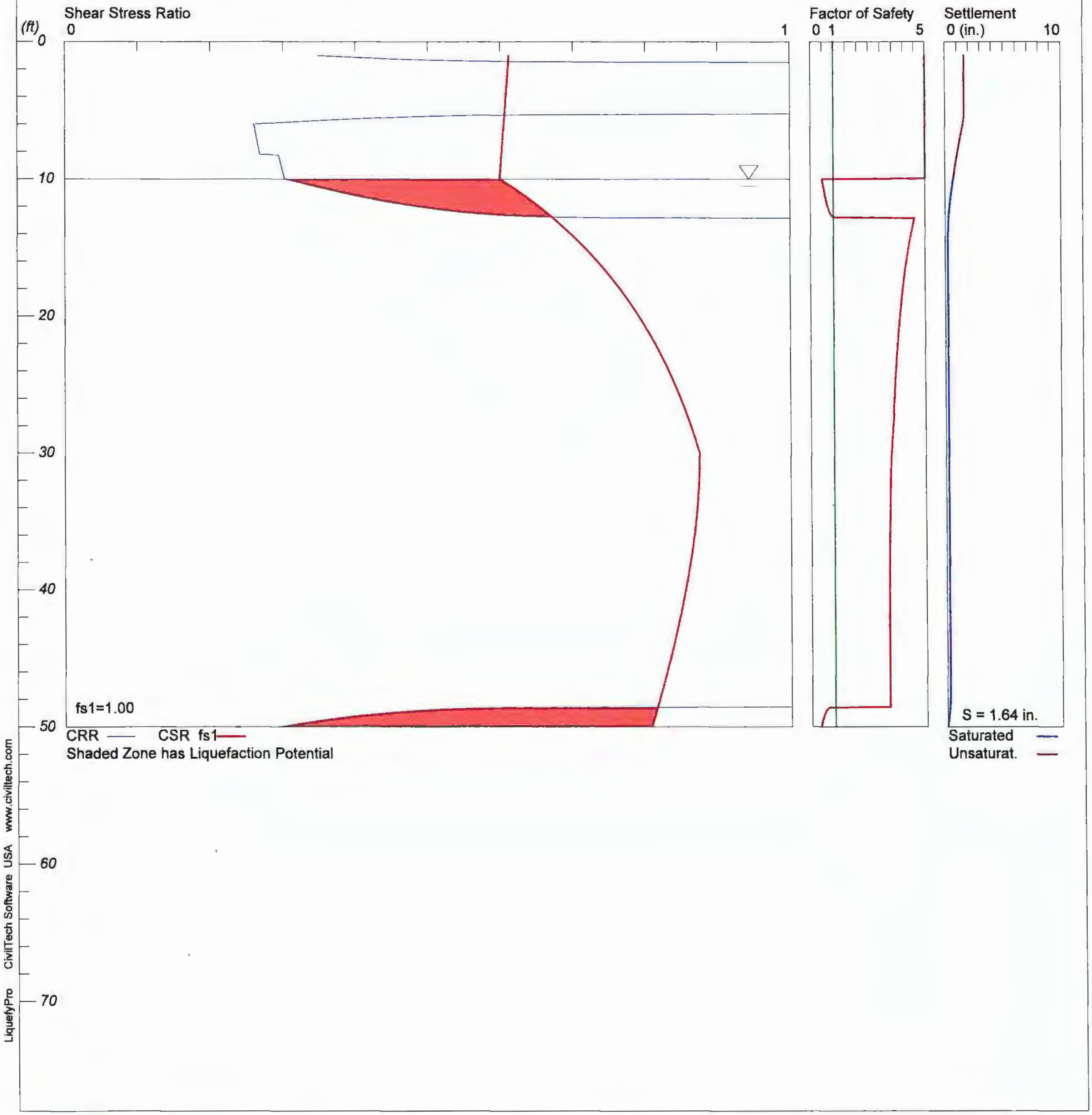
_________________________________________________________________________________
_
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with
user request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```

LIQUEFACTION ANALYSIS

Boring B-1 - Full PGA ; 2,475 yr Rtrn; FS 1.0

Hole No.=B-1   Water Depth=10 ft   Surface Elev.=96 ft

Magnitude=6.36
Acceleration=0.944g



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***********************************************************************************
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                                          LIQUEFACTION ANALYSIS SUMMARY

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   2/20/2024       4:28:45 PM

        Input File Name: C:\Users\jmaier\Desktop\Culver City CBC 2022 Liq Full PGA
- 2475 Rtrn - FS 1.0.liq
        Title:  Boring B-1 - Full PGA ; 2,475 yr Rtrn; FS 1.0
        Subtitle:  PS Los Angeles CA – CBC 2022

        Surface Elev.=96 ft
        Hole No.=B-1
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration= 0.94 g
        Earthquake Magnitude= 6.36

 Input Data:
        Surface Elev.=96 ft
        Hole No.=B-1
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration=0.94 g
        Earthquake Magnitude=6.36
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu/Seed
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                    Ce = 1.25
        7. Borehole Diameter,                                          Cb= 1
        8. Sampling Method,                                           Cs= 1.2
        9. User request factor of safety (apply to CSR) ,   User= 1.0
           Plot one CSR curve (fs1=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options
```

```
        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ___________________________________
        1.00    7.00    120.00  70.00
        3.50    28.00   120.00  30.00
        6.00    5.00    120.00  30.00
        10.00   7.00    120.00  30.00
        15.00   20.00   120.00  35.00
        20.00   41.00   120.00  14.00
        25.00   88.00   120.00  10.00
        30.00   55.00   120.00  10.00
        35.00   86.00   120.00  10.00
        40.00   100.00  120.00  10.00
        45.00   46.00   120.00  10.00
        50.00   14.00   120.00  NoLiq
        ___________________________________

Output Results:
        Settlement of Saturated Sands=0.71 in.
        Settlement of Unsaturated Sands=0.93 in.
        Total Settlement of Saturated and Unsaturated Sands=1.64 in.
        Differential Settlement=0.818 to 1.080 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        1.00    0.35    0.61    5.00    0.71    0.93    1.64
        1.50    3.05    0.61    5.00    0.71    0.93    1.63
        2.00    3.05    0.61    5.00    0.71    0.92    1.63
        2.50    3.05    0.61    5.00    0.71    0.92    1.63
        3.00    3.05    0.61    5.00    0.71    0.92    1.63
        3.50    3.05    0.61    5.00    0.71    0.92    1.63
        4.00    3.05    0.61    5.00    0.71    0.92    1.63
        4.50    3.05    0.61    5.00    0.71    0.92    1.63
        5.00    3.05    0.61    5.00    0.71    0.91    1.62
        5.50    0.45    0.61    5.00    0.71    0.90    1.61
        6.00    0.26    0.61    5.00    0.71    0.82    1.53
        6.50    0.26    0.60    5.00    0.71    0.71    1.41
        7.00    0.26    0.60    5.00    0.71    0.59    1.30
        7.50    0.27    0.60    5.00    0.71    0.48    1.19
        8.00    0.27    0.60    5.00    0.71    0.37    1.07
        8.50    0.30    0.60    5.00    0.71    0.26    0.97
        9.00    0.30    0.60    5.00    0.71    0.16    0.87
        9.50    0.30    0.60    5.00    0.71    0.07    0.77
        10.00   0.30    0.60    0.50*   0.71    0.00    0.71
        10.50   0.34    0.61    0.56*   0.62    0.00    0.62
        11.00   0.38    0.63    0.61*   0.53    0.00    0.53
        11.50   0.43    0.64    0.67*   0.46    0.00    0.46
        12.00   0.48    0.65    0.74*   0.39    0.00    0.39
```

12.50	0.56	0.66	0.85*	0.33	0.00	0.33
13.00	3.05	0.68	4.51	0.28	0.00	0.28
13.50	3.05	0.69	4.44	0.25	0.00	0.25
14.00	3.05	0.70	4.37	0.23	0.00	0.23
14.50	3.05	0.71	4.31	0.22	0.00	0.22
15.00	3.05	0.72	4.26	0.22	0.00	0.22
15.50	3.05	0.73	4.20	0.22	0.00	0.22
16.00	3.05	0.73	4.16	0.22	0.00	0.22
16.50	3.05	0.74	4.11	0.22	0.00	0.22
17.00	3.05	0.75	4.07	0.22	0.00	0.22
17.50	3.05	0.76	4.03	0.22	0.00	0.22
18.00	3.05	0.76	3.99	0.22	0.00	0.22
18.50	3.05	0.77	3.95	0.22	0.00	0.22
19.00	3.05	0.78	3.92	0.22	0.00	0.22
19.50	3.05	0.78	3.89	0.22	0.00	0.22
20.00	3.05	0.79	3.86	0.22	0.00	0.22
20.50	3.05	0.80	3.83	0.22	0.00	0.22
21.00	3.05	0.80	3.80	0.22	0.00	0.22
21.50	3.05	0.81	3.78	0.22	0.00	0.22
22.00	3.05	0.81	3.75	0.22	0.00	0.22
22.50	3.05	0.82	3.73	0.22	0.00	0.22
23.00	3.05	0.82	3.71	0.22	0.00	0.22
23.50	3.05	0.83	3.69	0.22	0.00	0.22
24.00	3.05	0.83	3.67	0.22	0.00	0.22
24.50	3.05	0.84	3.65	0.22	0.00	0.22
25.00	3.05	0.84	3.63	0.22	0.00	0.22
25.50	3.05	0.84	3.61	0.22	0.00	0.22
26.00	3.05	0.85	3.60	0.22	0.00	0.22
26.50	3.05	0.85	3.58	0.22	0.00	0.22
27.00	3.05	0.85	3.57	0.22	0.00	0.22
27.50	3.06	0.86	3.57	0.22	0.00	0.22
28.00	3.05	0.86	3.54	0.22	0.00	0.22
28.50	3.04	0.86	3.52	0.22	0.00	0.22
29.00	3.03	0.87	3.50	0.22	0.00	0.22
29.50	3.03	0.87	3.48	0.22	0.00	0.22
30.00	3.02	0.87	3.45	0.22	0.00	0.22
30.50	3.01	0.87	3.45	0.22	0.00	0.22
31.00	3.00	0.87	3.44	0.22	0.00	0.22
31.50	2.99	0.87	3.43	0.22	0.00	0.22
32.00	2.98	0.87	3.42	0.22	0.00	0.22
32.50	2.97	0.87	3.41	0.22	0.00	0.22
33.00	2.97	0.87	3.40	0.22	0.00	0.22
33.50	2.96	0.87	3.40	0.22	0.00	0.22
34.00	2.95	0.87	3.39	0.22	0.00	0.22
34.50	2.94	0.87	3.38	0.22	0.00	0.22
35.00	2.93	0.87	3.38	0.22	0.00	0.22
35.50	2.93	0.87	3.37	0.22	0.00	0.22
36.00	2.92	0.87	3.37	0.22	0.00	0.22
36.50	2.91	0.86	3.37	0.22	0.00	0.22
37.00	2.90	0.86	3.36	0.22	0.00	0.22

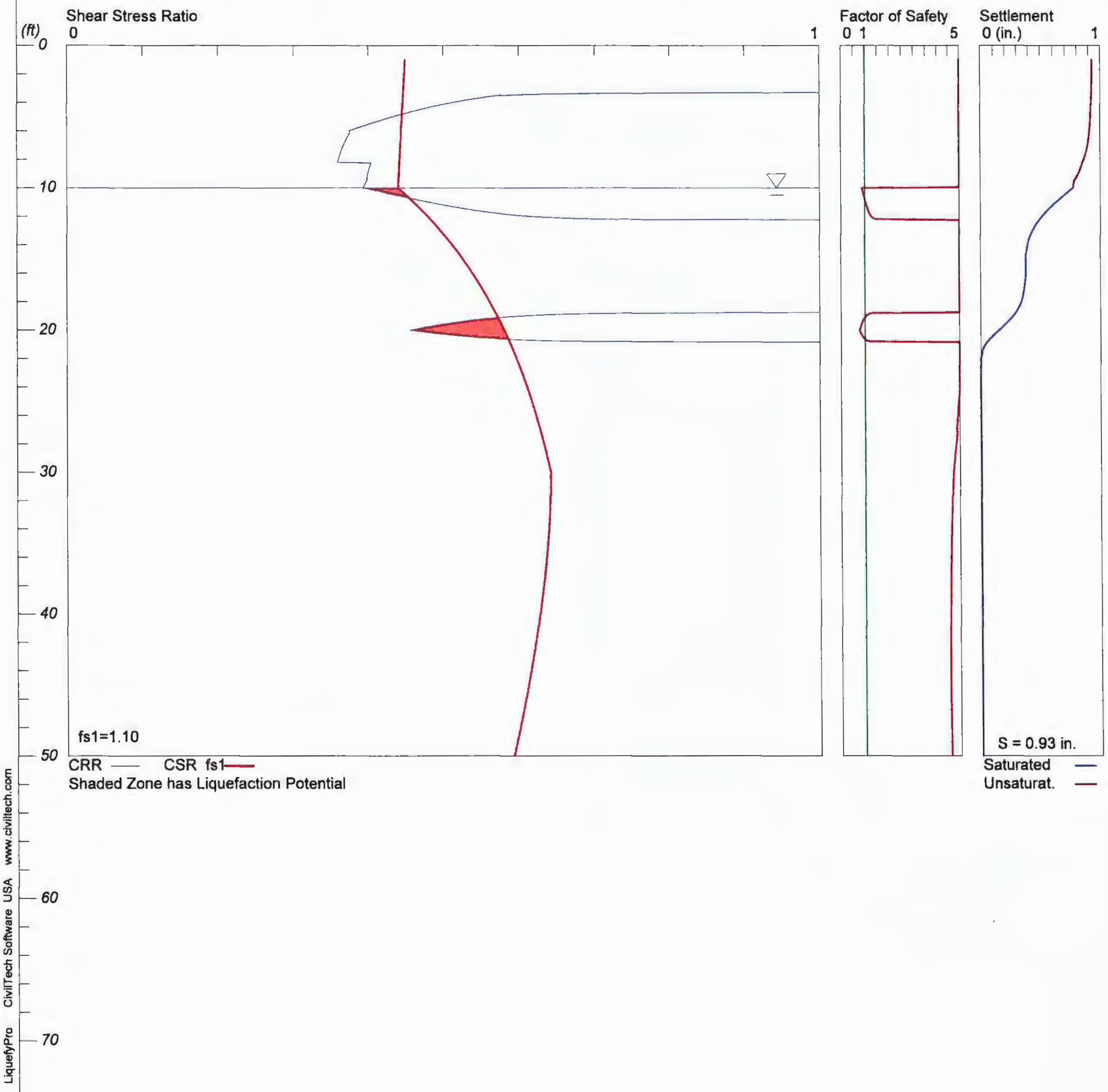
```
        37.50   2.89    0.86    3.36    0.22    0.00    0.22
        38.00   2.89    0.86    3.35    0.22    0.00    0.22
        38.50   2.88    0.86    3.35    0.22    0.00    0.22
        39.00   2.87    0.86    3.35    0.22    0.00    0.22
        39.50   2.86    0.86    3.35    0.22    0.00    0.22
        40.00   2.85    0.85    3.34    0.22    0.00    0.22
        40.50   2.85    0.85    3.34    0.22    0.00    0.22
        41.00   2.84    0.85    3.34    0.22    0.00    0.22
        41.50   2.83    0.85    3.34    0.22    0.00    0.22
        42.00   2.82    0.85    3.34    0.22    0.00    0.22
        42.50   2.82    0.84    3.34    0.22    0.00    0.22
        43.00   2.81    0.84    3.34    0.22    0.00    0.22
        43.50   2.80    0.84    3.34    0.22    0.00    0.22
        44.00   2.80    0.84    3.34    0.22    0.00    0.22
        44.50   2.79    0.83    3.34    0.22    0.00    0.22
        45.00   2.78    0.83    3.34    0.22    0.00    0.22
        45.50   2.77    0.83    3.34    0.22    0.00    0.22
        46.00   2.77    0.83    3.34    0.22    0.00    0.22
        46.50   2.76    0.82    3.35    0.22    0.00    0.22
        47.00   2.75    0.82    3.35    0.22    0.00    0.22
        47.50   2.75    0.82    3.35    0.22    0.00    0.22
        48.00   2.74    0.82    3.35    0.22    0.00    0.22
        48.50   2.73    0.81    3.36    0.20    0.00    0.20
        49.00   0.44    0.81    0.54*   0.14    0.00    0.14
        49.50   0.35    0.81    0.44*   0.07    0.00    0.07
        50.00   0.29    0.81    0.36*   0.00    0.00    0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

____________________________________________________________________________________
_
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with
user request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```

LIQUEFACTION ANALYSIS

Boring B-6 - 2/3 PGA ; 475 yr rtrn FS 1.1

Hole No.=B-1* *Water Depth=10 ft* *Surface Elev.=98 ft

Magnitude=6.35
Acceleration=0.63g



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                                          LIQUEFACTION ANALYSIS SUMMARY

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   2/20/2024       4:25:38 PM

        Input File Name: C:\Users\jmaier\Desktop\B-6   Culver City CBC 2022 Liq 2-3
PGA - 475 Rtrn - FS 1.1.liq
        Title:  Boring B-6 - 2/3  PGA ; 475 yr rtrn FS 1.1
        Subtitle:  PS Los Angeles CA – CBC 2022

        Surface Elev.=98 ft
        Hole No.=B-1
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration= 0.63 g
        Earthquake Magnitude= 6.35

 Input Data:
        Surface Elev.=98 ft
        Hole No.=B-1
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration=0.63 g
        Earthquake Magnitude=6.35
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu/Seed
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                    Ce = 1.25
        7. Borehole Diameter,                                          Cb= 1
        8. Sampling Method,                                           Cs= 1.2
        9. User request factor of safety (apply to CSR) ,   User= 1.1
           Plot one CSR curve (fs1=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options
```

```
        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        1.00    19.00   120.00  30.00
        3.50    11.00   120.00  30.00
        6.00    8.00    120.00  30.00
        10.00   9.00    120.00  36.00
        15.00   22.00   120.00  16.00
        20.00   13.00   120.00  45.00
        25.00   31.00   120.00  45.00
        30.00   62.00   120.00  10.00
        35.00   67.00   120.00  18.00
        40.00   67.00   120.00  24.00
        45.00   63.00   120.00  30.00
        50.00   84.00   120.00  30.00
        ____________________________________

Output Results:
        Settlement of Saturated Sands=0.77 in.
        Settlement of Unsaturated Sands=0.16 in.
        Total Settlement of Saturated and Unsaturated Sands=0.93 in.
        Differential Settlement=0.467 to 0.616 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        ________________________________________________________
        1.00    3.06    0.45    5.00    0.77    0.16    0.93
        1.50    3.06    0.45    5.00    0.77    0.16    0.93
        2.00    3.06    0.45    5.00    0.77    0.16    0.93
        2.50    3.06    0.45    5.00    0.77    0.16    0.93
        3.00    3.06    0.45    5.00    0.77    0.16    0.93
        3.50    0.57    0.45    5.00    0.77    0.16    0.93
        4.00    0.51    0.45    5.00    0.77    0.15    0.93
        4.50    0.47    0.45    5.00    0.77    0.15    0.92
        5.00    0.43    0.45    5.00    0.77    0.15    0.92
        5.50    0.40    0.44    5.00    0.77    0.14    0.92
        6.00    0.37    0.44    5.00    0.77    0.14    0.91
        6.50    0.37    0.44    5.00    0.77    0.13    0.91
        7.00    0.37    0.44    5.00    0.77    0.12    0.90
        7.50    0.36    0.44    5.00    0.77    0.11    0.88
        8.00    0.36    0.44    5.00    0.77    0.09    0.86
        8.50    0.40    0.44    5.00    0.77    0.07    0.84
        9.00    0.40    0.44    5.00    0.77    0.04    0.82
        9.50    0.40    0.44    5.00    0.77    0.01    0.78
        10.00   0.39    0.44    0.89*   0.77    0.00    0.77
        10.50   0.44    0.45    0.97*   0.70    0.00    0.70
        11.00   0.48    0.46    1.04    0.63    0.00    0.63
        11.50   0.53    0.47    1.14    0.57    0.00    0.57
        12.00   0.62    0.48    1.29    0.52    0.00    0.52
```

12.50	3.06	0.49	5.00	0.47	0.00	0.47
13.00	3.06	0.50	5.00	0.44	0.00	0.44
13.50	3.06	0.50	5.00	0.41	0.00	0.41
14.00	3.06	0.51	5.00	0.40	0.00	0.40
14.50	3.06	0.52	5.00	0.39	0.00	0.39
15.00	3.06	0.53	5.00	0.38	0.00	0.38
15.50	3.06	0.53	5.00	0.38	0.00	0.38
16.00	3.06	0.54	5.00	0.38	0.00	0.38
16.50	3.06	0.54	5.00	0.38	0.00	0.38
17.00	3.06	0.55	5.00	0.37	0.00	0.37
17.50	3.06	0.56	5.00	0.36	0.00	0.36
18.00	3.06	0.56	5.00	0.34	0.00	0.34
18.50	3.06	0.57	5.00	0.31	0.00	0.31
19.00	0.60	0.57	1.05	0.27	0.00	0.27
19.50	0.51	0.58	0.89*	0.22	0.00	0.22
20.00	0.46	0.58	0.78*	0.15	0.00	0.15
20.50	0.55	0.58	0.93*	0.09	0.00	0.09
21.00	3.06	0.59	5.00	0.04	0.00	0.04
21.50	3.06	0.59	5.00	0.01	0.00	0.01
22.00	3.06	0.60	5.00	0.00	0.00	0.00
22.50	3.06	0.60	5.00	0.00	0.00	0.00
23.00	3.06	0.60	5.00	0.00	0.00	0.00
23.50	3.06	0.61	5.00	0.00	0.00	0.00
24.00	3.06	0.61	5.00	0.00	0.00	0.00
24.50	3.06	0.61	4.99	0.00	0.00	0.00
25.00	3.06	0.62	4.97	0.00	0.00	0.00
25.50	3.06	0.62	4.94	0.00	0.00	0.00
26.00	3.06	0.62	4.92	0.00	0.00	0.00
26.50	3.06	0.62	4.90	0.00	0.00	0.00
27.00	3.06	0.63	4.88	0.00	0.00	0.00
27.50	3.07	0.63	4.88	0.00	0.00	0.00
28.00	3.06	0.63	4.85	0.00	0.00	0.00
28.50	3.06	0.63	4.81	0.00	0.00	0.00
29.00	3.05	0.64	4.78	0.00	0.00	0.00
29.50	3.04	0.64	4.75	0.00	0.00	0.00
30.00	3.03	0.64	4.72	0.00	0.00	0.00
30.50	3.02	0.64	4.71	0.00	0.00	0.00
31.00	3.01	0.64	4.70	0.00	0.00	0.00
31.50	3.00	0.64	4.69	0.00	0.00	0.00
32.00	3.00	0.64	4.68	0.00	0.00	0.00
32.50	2.99	0.64	4.67	0.00	0.00	0.00
33.00	2.98	0.64	4.66	0.00	0.00	0.00
33.50	2.97	0.64	4.65	0.00	0.00	0.00
34.00	2.96	0.64	4.64	0.00	0.00	0.00
34.50	2.95	0.64	4.63	0.00	0.00	0.00
35.00	2.95	0.64	4.62	0.00	0.00	0.00
35.50	2.94	0.64	4.62	0.00	0.00	0.00
36.00	2.93	0.64	4.61	0.00	0.00	0.00
36.50	2.92	0.63	4.60	0.00	0.00	0.00
37.00	2.91	0.63	4.60	0.00	0.00	0.00

```
        37.50   2.91    0.63    4.59    0.00    0.00    0.00
        38.00   2.90    0.63    4.59    0.00    0.00    0.00
        38.50   2.89    0.63    4.58    0.00    0.00    0.00
        39.00   2.88    0.63    4.58    0.00    0.00    0.00
        39.50   2.87    0.63    4.58    0.00    0.00    0.00
        40.00   2.87    0.63    4.57    0.00    0.00    0.00
        40.50   2.86    0.63    4.57    0.00    0.00    0.00
        41.00   2.85    0.62    4.57    0.00    0.00    0.00
        41.50   2.84    0.62    4.57    0.00    0.00    0.00
        42.00   2.84    0.62    4.57    0.00    0.00    0.00
        42.50   2.83    0.62    4.57    0.00    0.00    0.00
        43.00   2.82    0.62    4.57    0.00    0.00    0.00
        43.50   2.81    0.62    4.57    0.00    0.00    0.00
        44.00   2.81    0.61    4.57    0.00    0.00    0.00
        44.50   2.80    0.61    4.57    0.00    0.00    0.00
        45.00   2.79    0.61    4.57    0.00    0.00    0.00
        45.50   2.79    0.61    4.57    0.00    0.00    0.00
        46.00   2.78    0.61    4.57    0.00    0.00    0.00
        46.50   2.77    0.61    4.58    0.00    0.00    0.00
        47.00   2.76    0.60    4.58    0.00    0.00    0.00
        47.50   2.76    0.60    4.58    0.00    0.00    0.00
        48.00   2.75    0.60    4.59    0.00    0.00    0.00
        48.50   2.74    0.60    4.59    0.00    0.00    0.00
        49.00   2.74    0.60    4.59    0.00    0.00    0.00
        49.50   2.73    0.59    4.60    0.00    0.00    0.00
        50.00   2.72    0.59    4.60    0.00    0.00    0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

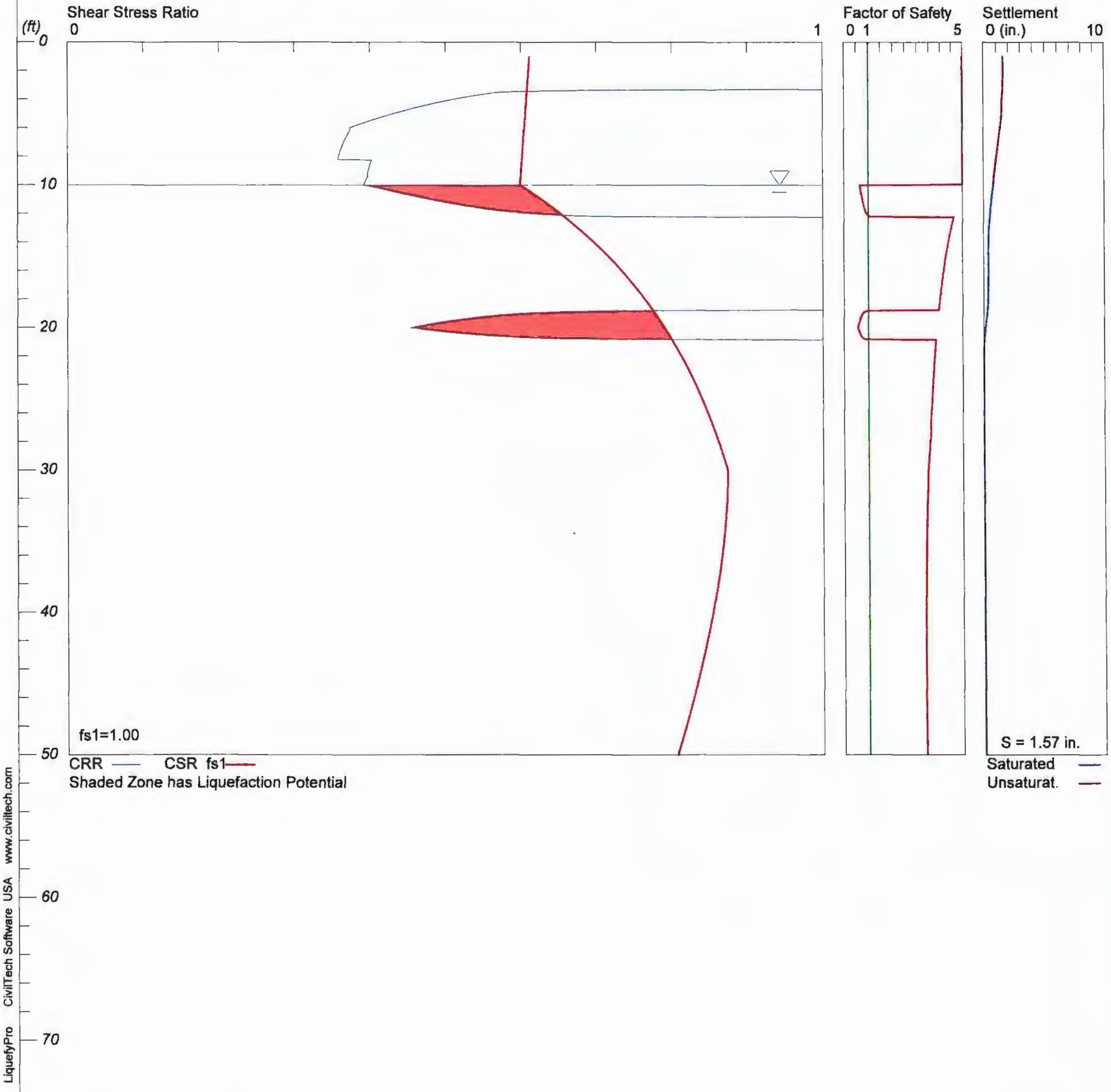
___________________________________________________________________________________
_
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with
user request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```

LIQUEFACTION ANALYSIS

Boring B-6 - Full PGA ; 2,475 yr Rtrn; FS 1.0

Hole No.=B-6   Water Depth=10 ft   Surface Elev.=96 ft

Magnitude=6.36
Acceleration=0.944g



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******************************************************************************************
********************
                                             LIQUEFACTION ANALYSIS SUMMARY

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   2/20/2024       4:23:30 PM

        Input File Name: C:\Users\jmaier\Desktop\B-6   Culver City CBC 2022 Liq
Full PGA - 2475 Rtrn - FS 1.0.liq
        Title:  Boring B-6 - Full PGA ; 2,475 yr Rtrn; FS 1.0
        Subtitle:   PS Los Angeles CA – CBC 2022

        Surface Elev.=96 ft
        Hole No.=B-6
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration= 0.94 g
        Earthquake Magnitude= 6.36

 Input Data:
        Surface Elev.=96 ft
        Hole No.=B-6
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 10.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration=0.94 g
        Earthquake Magnitude=6.36
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Tokimatsu/Seed
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                 Ce = 1.25
        7. Borehole Diameter,                                      Cb= 1
        8. Sampling Method,                                       Cs= 1.2
        9. User request factor of safety (apply to CSR) ,   User= 1.0
           Plot one CSR curve (fs1=User)
        10. Use Curve Smoothing: Yes*
        * Recommended Options
```

```
        In-Situ Test Data:
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        1.00    19.00   120.00  30.00
        3.50    11.00   120.00  30.00
        6.00    8.00    120.00  30.00
        10.00   9.00    120.00  36.00
        15.00   22.00   120.00  16.00
        20.00   13.00   120.00  45.00
        25.00   31.00   120.00  45.00
        30.00   62.00   120.00  10.00
        35.00   67.00   120.00  18.00
        40.00   67.00   120.00  24.00
        45.00   63.00   120.00  30.00
        50.00   84.00   120.00  30.00
        ____________________________________

Output Results:
        Settlement of Saturated Sands=0.78 in.
        Settlement of Unsaturated Sands=0.79 in.
        Total Settlement of Saturated and Unsaturated Sands=1.57 in.
        Differential Settlement=0.783 to 1.034 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        1.00    3.05    0.61    5.00    0.78    0.79    1.57
        1.50    3.05    0.61    5.00    0.78    0.79    1.57
        2.00    3.05    0.61    5.00    0.78    0.79    1.57
        2.50    3.05    0.61    5.00    0.78    0.79    1.56
        3.00    3.05    0.61    5.00    0.78    0.78    1.56
        3.50    0.57    0.61    5.00    0.78    0.75    1.53
        4.00    0.51    0.61    5.00    0.78    0.70    1.48
        4.50    0.47    0.61    5.00    0.78    0.68    1.46
        5.00    0.43    0.61    5.00    0.78    0.66    1.44
        5.50    0.40    0.61    5.00    0.78    0.61    1.39
        6.00    0.37    0.61    5.00    0.78    0.54    1.32
        6.50    0.37    0.60    5.00    0.78    0.47    1.25
        7.00    0.37    0.60    5.00    0.78    0.39    1.17
        7.50    0.36    0.60    5.00    0.78    0.32    1.09
        8.00    0.36    0.60    5.00    0.78    0.24    1.02
        8.50    0.40    0.60    5.00    0.78    0.17    0.95
        9.00    0.40    0.60    5.00    0.78    0.10    0.88
        9.50    0.40    0.60    5.00    0.78    0.03    0.81
        10.00   0.39    0.60    0.65*   0.78    0.00    0.78
        10.50   0.43    0.61    0.71*   0.70    0.00    0.70
        11.00   0.48    0.63    0.76*   0.64    0.00    0.64
        11.50   0.53    0.64    0.83*   0.57    0.00    0.57
        12.00   0.61    0.65    0.94*   0.52    0.00    0.52
```

12.50	3.05	0.66	4.59	0.47	0.00	0.47
13.00	3.05	0.68	4.51	0.44	0.00	0.44
13.50	3.05	0.69	4.44	0.41	0.00	0.41
14.00	3.05	0.70	4.37	0.40	0.00	0.40
14.50	3.05	0.71	4.31	0.39	0.00	0.39
15.00	3.05	0.72	4.26	0.38	0.00	0.38
15.50	3.05	0.73	4.20	0.38	0.00	0.38
16.00	3.05	0.73	4.16	0.38	0.00	0.38
16.50	3.05	0.74	4.11	0.38	0.00	0.38
17.00	3.05	0.75	4.07	0.37	0.00	0.37
17.50	3.05	0.76	4.03	0.36	0.00	0.36
18.00	3.05	0.76	3.99	0.34	0.00	0.34
18.50	3.05	0.77	3.95	0.31	0.00	0.31
19.00	0.60	0.78	0.77*	0.27	0.00	0.27
19.50	0.51	0.78	0.65*	0.22	0.00	0.22
20.00	0.45	0.79	0.57*	0.15	0.00	0.15
20.50	0.54	0.80	0.68*	0.09	0.00	0.09
21.00	3.05	0.80	3.80	0.04	0.00	0.04
21.50	3.05	0.81	3.78	0.01	0.00	0.01
22.00	3.05	0.81	3.75	0.00	0.00	0.00
22.50	3.05	0.82	3.73	0.00	0.00	0.00
23.00	3.05	0.82	3.71	0.00	0.00	0.00
23.50	3.05	0.83	3.69	0.00	0.00	0.00
24.00	3.05	0.83	3.67	0.00	0.00	0.00
24.50	3.05	0.84	3.65	0.00	0.00	0.00
25.00	3.05	0.84	3.63	0.00	0.00	0.00
25.50	3.05	0.84	3.61	0.00	0.00	0.00
26.00	3.05	0.85	3.60	0.00	0.00	0.00
26.50	3.05	0.85	3.58	0.00	0.00	0.00
27.00	3.05	0.85	3.57	0.00	0.00	0.00
27.50	3.06	0.86	3.57	0.00	0.00	0.00
28.00	3.05	0.86	3.54	0.00	0.00	0.00
28.50	3.04	0.86	3.52	0.00	0.00	0.00
29.00	3.03	0.87	3.50	0.00	0.00	0.00
29.50	3.03	0.87	3.48	0.00	0.00	0.00
30.00	3.02	0.87	3.45	0.00	0.00	0.00
30.50	3.01	0.87	3.45	0.00	0.00	0.00
31.00	3.00	0.87	3.44	0.00	0.00	0.00
31.50	2.99	0.87	3.43	0.00	0.00	0.00
32.00	2.98	0.87	3.42	0.00	0.00	0.00
32.50	2.97	0.87	3.41	0.00	0.00	0.00
33.00	2.97	0.87	3.40	0.00	0.00	0.00
33.50	2.96	0.87	3.40	0.00	0.00	0.00
34.00	2.95	0.87	3.39	0.00	0.00	0.00
34.50	2.94	0.87	3.38	0.00	0.00	0.00
35.00	2.93	0.87	3.38	0.00	0.00	0.00
35.50	2.93	0.87	3.37	0.00	0.00	0.00
36.00	2.92	0.87	3.37	0.00	0.00	0.00
36.50	2.91	0.86	3.37	0.00	0.00	0.00
37.00	2.90	0.86	3.36	0.00	0.00	0.00

```
        37.50    2.89     0.86     3.36     0.00     0.00     0.00
        38.00    2.89     0.86     3.35     0.00     0.00     0.00
        38.50    2.88     0.86     3.35     0.00     0.00     0.00
        39.00    2.87     0.86     3.35     0.00     0.00     0.00
        39.50    2.86     0.86     3.35     0.00     0.00     0.00
        40.00    2.85     0.85     3.34     0.00     0.00     0.00
        40.50    2.85     0.85     3.34     0.00     0.00     0.00
        41.00    2.84     0.85     3.34     0.00     0.00     0.00
        41.50    2.83     0.85     3.34     0.00     0.00     0.00
        42.00    2.82     0.85     3.34     0.00     0.00     0.00
        42.50    2.82     0.84     3.34     0.00     0.00     0.00
        43.00    2.81     0.84     3.34     0.00     0.00     0.00
        43.50    2.80     0.84     3.34     0.00     0.00     0.00
        44.00    2.80     0.84     3.34     0.00     0.00     0.00
        44.50    2.79     0.83     3.34     0.00     0.00     0.00
        45.00    2.78     0.83     3.34     0.00     0.00     0.00
        45.50    2.77     0.83     3.34     0.00     0.00     0.00
        46.00    2.77     0.83     3.34     0.00     0.00     0.00
        46.50    2.76     0.82     3.35     0.00     0.00     0.00
        47.00    2.75     0.82     3.35     0.00     0.00     0.00
        47.50    2.75     0.82     3.35     0.00     0.00     0.00
        48.00    2.74     0.82     3.35     0.00     0.00     0.00
        48.50    2.73     0.81     3.36     0.00     0.00     0.00
        49.00    2.73     0.81     3.36     0.00     0.00     0.00
        49.50    2.72     0.81     3.36     0.00     0.00     0.00
        50.00    2.71     0.81     3.37     0.00     0.00     0.00
        _______________________________________________________
        * F.S.<1, Liquefaction Potential Zone
        (F.S. is limited to 5,  CRR is limited to 2,    CSR is limited to 2)

        Units:  Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =
pcf; Depth = ft; Settlement = in.

_________________________________________________________________________________
_
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake (with
user request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```

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.

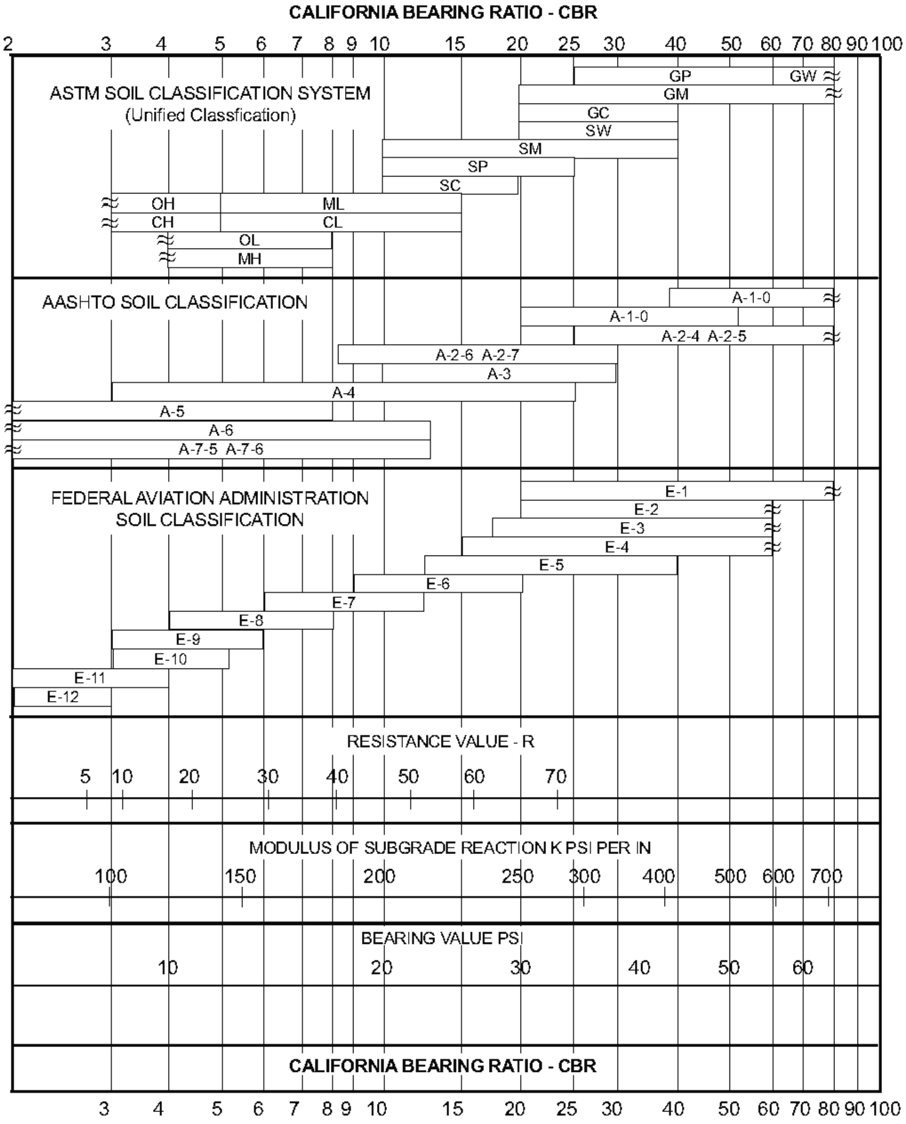
Laboratory Testing

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.

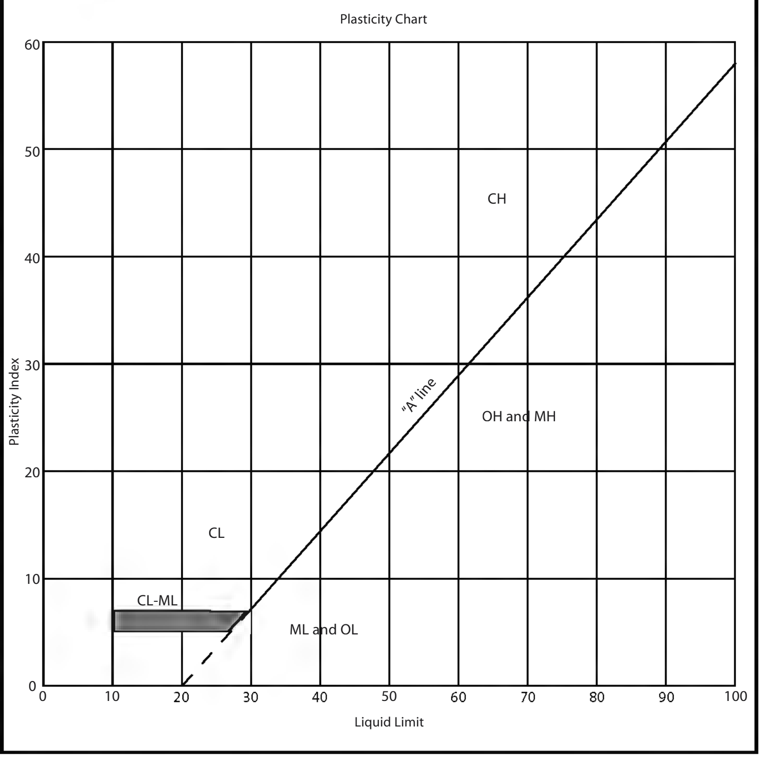
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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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