APPENDIX D.

Geotechnical Feasibility Study

YELLOW

Geotechnical Feasibility Study

Proposed Public Storage 3-Story Building
12681 W. Jefferson Boulevard
Los Angeles, California

Prepared for:

Public Storage
Glendale, California

September 14, 2018
Project No. 2G-1806003

GILES
ENGINEERING ASSOCIATES, INC.

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

- Atlanta, GA
- Baltimore, MD
- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

September 14, 2018

Public Storage, Inc.
701 Western Avenue
Glendale California 91201

Attention: Mr. Andres Friedman
Vice President, Development, Real Estate Group

Subject: Geotechnical Feasibility Study
Proposed Public Storage 3-Story Building
12681 W. Jefferson Boulevard
Los Angeles, California
Project No. 2G-1806003

Dear Mr. Friedman:

In accordance with your request and authorization, a *Geotechnical Feasibility Study* 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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12681 W. JEFFERNSON BOULEVARD
LOS ANGELES, CALIFORNIA
PROJECT NO. 2G-1806003

GEOTECHNICAL FEASIBILITY STUDY

PROPOED PUBLIC STORAGE 3-STORY BUILDING
12681 W. JEFFERSON BOULEVARD
LOS ANGELES, CALIFORNIA
PROJECT NO. 2G-1806003

EXECUTIVE SUMMARY OUTLINE

Subsurface Conditions

- Site Class designation D is recommended for seismic design considerations.
- Our review of the *Geologic Map of the Venice Quadrangle* within the Seismic Hazard Zone Report for the Venice Quadrangle indicated that the subject site is underlain by Young Alluvial Fan Deposits consisting generally of fine to medium grained sand with silt and clay layers.
- Fill and possible fill materials were encountered within our exploratory test borings to depths of about 3½ to 5 feet below existing grades. These materials were generally noted to be damp to moist, medium stiff sandy silt, and clayey silt, and damp to very moist, loose to firm, clayey sand and silty sand.
- Native soils encountered underneath the fill and possible fill material generally consisted of alternating layers of sand, silt, and clay, with variable relative densities and comparative consistencies.
- Groundwater was encountered within our test borings during our subsurface investigation at depths of 20 to 25 feet below grade. The historic high groundwater elevation for the site is about 7 feet below grade as determined from the maps available in the Seismic Hazard Zone Report for this area.

Site Development

- Proposed site development will include the construction of a new three-story building with no basement or below grade structures.
- Due to the presence of undocumented fill and possible fill and the variable strength characteristics of the near surface on-site soils, some grading and over-excavation will be required for development of new structures. Over-excavation is anticipated to extend to maximum depths of about 5 feet below existing ground surfaces, and at least 2 feet below the bottom of foundations, whichever is greater. This preliminary estimated overexcavation of the existing fill and possible fill is considered the maximum required amount of removal of undocumented fill per City of Los Angeles requirements. The depth of required removal should be confirmed with additional field testing including shallow borings and test pits.

Building Foundation

- The proposed structure may be supported by a mat/slab supported on a minimum 2 foot thick structural compacted fill layer designed for a maximum, net allowable soil bearing pressure of 2,000 to 3,000 pounds per square foot (psf) or a maximum modulus of subgrade reaction (k_s) of 50 to 70 pounds per square inch per inch (psi/in.)
- Steel reinforcing should be design by the structural engineer.

Building Floor Slab

- The ground floor of the new building may be designed as a load-bearing mat/slab.
- The mat/slab should be underlain by a minimum 4-inch thick granular base supported on a properly prepared subgrade consisting of newly placed structural compacted fill.
- A minimum 15-mil vapor retarder is recommended below the floor slab or base course to protect moisture sensitive floor coverings.

Pavement

- Asphaltic Concrete: 3 inches of asphaltic concrete underlain by 7 or 10 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.

Special Considerations

- Some of the near surface soils have in-situ moisture contents that were very moist and above the optimum moisture content to an extent that may create difficulties in achieving the required compaction. Therefore, significant air-drying or import soil may be required.
- Additional field exploration, possibly including both shallow borings and backhoe test pits is recommended to better document the depth and extent of existing fill requiring removal within the proposed building area.
- The site is located within a zone requiring a methane gas study. The additional borings will be converted to methane test holes to perform this study.

YELLOW – This site has been given a yellow designation due to increased costs associated with the removal of about 5 feet of undocumented fill per City of Los Angeles requirements, the grading and compaction of the overly moist, near surface soil, and requirement for significant air-drying or import soil.

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Feasibility Study* that Giles Engineering Associates, Inc. ("Giles") conducted at the subject site. This study 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 future site development. The scope of each service area is briefly explained in this report.

The purpose of this geotechnical study is to evaluate subsurface soil conditions and to provide preliminary geotechnical recommendations for the design and construction of future site development. Our report includes a description of the work performed at the site, a discussion of the geotechnical condition and concerns observed, and recommendations developed from our engineering analysis of field and laboratory data.

Giles has not conducted a Phase 1 Environmental Site Assessment for the subject site. Giles offers these services. Please contact our office if these services are needed.

2.0 SITES AND PROJECT DESCRIPTION

The project includes a new 3-story building with new drive and parking areas. No basement or underground structure is planned. The site is located at 12681 West Jefferson Boulevard, in the City of Los Angeles, California. The western area of the site is vacant with the eastern area developed with a 4-story, Public Storage building and parking lot outside the limits of the planned improvements. The site is bounded by apartment buildings to the north, commercial buildings to the west and east, and West Jefferson Boulevard to the south. For this report, Project North is considered perpendicular to West Jefferson Boulevard.

Based on the site elevations estimated from Google Earth, elevations within the proposed building pad area range from about Elevation 17 feet at the southwestern corner of the proposed building to Elevation 23 in the center of the building. Therefore, cuts and fills of about 3 feet are expected to achieve the finish building pad elevation not considering possible overexcation requirements.

According to ALTA Map provided the site in the area of the building pad previously had a 2-story residence, barn and shed, in the southwest corner of the planned building. The site grades as determined from the ALTA Survey indicated the site slopes about 1 foot up toward the north and is level with the adjacent sidewalk to the south.

The new 3-story building is understood to be supported by perimeter bearing walls and interior walls and/or columns on a 10 x 10 foot grid. The exterior perimeter bearing wall is estimated to have a maximum load (total dead and live) of about 2 to 3 kips per lineal foot (kpf). The maximum loads for the interior columns are estimated to be in the range of 60 to 80 kips. Once structural loading information is known, that should be provided to Giles to determine if revisions to the recommendations contained in this report are necessary.

The traffic loading on the proposed parking lot improvement is understood to predominantly consist of automobiles with occasional heavy trucks. The parking lot pavement sections have been designed on the basis of daily traffic intensity equivalent to five equivalent 18-kip single axle loads and 1,500 automobiles within the main drive lanes and only automobiles of a lesser intensity within the parking stalls. Pavement designs are based on a 20-year design period. Therefore, the parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.5 for the automobile traffic parking stalls (light duty) and a TI of 5.5 for drive lane areas (medium to heavy duty). The existing parking lot area is generally in good conditions.

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of five (5) test borings (B-1 to B-5) to depths of 5 to 66 ½ feet below existing ground surfaces utilizing a truck mounted drill rig with hollow stem augers. The deeper borings (66 ½ feet) were performed to determine soil parameters at deeper layers and the potential for liquefaction at the subject site. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). Two of the borings, B-2 and B-5, were converted to percolation tests, P-2 and P-1, respectively. The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures and results 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 was performed in accordance with procedures in Appendix B of this report and included the collection of relatively undisturbed samples and bulk samples consisting of composite soil materials obtained at selected depth intervals from the borings. Relatively undisturbed samples were collected 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. Soil samples were also collected within the borings with Shelby tubes, 30 inches in length and 3 inches in diameter, in accordance with ASTM 1587.

Where deemed appropriate, 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 jars and transported to our laboratory for testing.

3.2 Subsurface Conditions

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

Soil

Our review of the *Geologic Map of the Venice Quadrangle* within the Seismic Hazard Zone Report for the Venice Quadrangle indicated that the subject site is underlain by Young Alluvial Fan Deposits consisting generally of fine to medium grained sand with silt and clay layers.

Fill and possible fill materials were encountered within our exploratory test borings to depths of about 3½ to 5 feet below existing grades. These materials were generally noted to be damp to moist, medium stiff sandy silt, and clayey silt, and damp to very moist, loose to firm, clayey sand and silty sand.

Native soils encountered underneath the fill and possible fill material generally consisted of alternating layers of sand, silt, and clay, with variable relative densities and comparative consistencies.

Groundwater

Groundwater was encountered within our test borings during our subsurface investigation at depths of 20 to 25 feet below grade. The historic high groundwater elevation for the site is about 7 feet below grade as determined from the maps available in the Seismic Hazard Zone Report for this area. Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

3.3 Percolation Testing

It is our understanding that an on-site below grade storm water infiltration system is being considered for the site. Two percolation tests (P-1 and P-2) were conducted at the northern and southern portions of the site (Figure 1) and involved the drilling of test borings about 8 inches in diameter to depths of about 5 feet below existing grade. Within the drilled test holes gravel, about 2 inches in thickness, was placed at the bottom of the test holes, then a two-inch diameter perforated pvc pipe was installed inside the boring and pea gravel was used as filter pack around the outside diameter of the pipe. Testing involved presoaking the test holes and then filling the test holes with water, and recording the drop in the water surface. After pre-soaking, successive test water was added to the casing to a minimum depth of 1 foot above the bottom of the casing and refilled to this level after each consecutive percolation test reading. The drop in water level over time is the pre-adjusted percolation rate at the test location. The pre-adjusted 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 infiltration rate.

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

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

The calculated infiltration rate was also adjusted to reflect a correction factor of 3 applied to the rate obtained from the infiltration test result and are summarized below. The calculated infiltration rates are summarized below.

TABLE 1 – PERCOLATION TEST RESULTS

Test Hole	**Test Depth^1 (feet)**	**Percolation Rate (in/hr)**	**Infiltration Rate ^{2} (in/hr)**	**Soil Type**
B-5/P-1	5.0	1.92	0.04	Silt, some fine Sand
B-2/P-2	5.0	0.96	0.08	Silty fine Sand

1) Depth is referenced to the existing surface grade at the test location.
2) Does not Reflects FS.

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.

4.0 LABORATORY TESTING

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

In Situ Moisture and Density

Tests were performed on select samples from the test borings to determine the subsoils dry density and natural moisture contents in accordance with Test Method ASTM 2216. The results of these tests indicated generally moist conditions on the upper five feet. In place moisture and density test results are included in the Test Boring Logs enclosed in Appendix A.

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.

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Boring B-1, B-3, B-4 and B-5 (0 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829. The result of our expansion index (EI) test indicates that the near surface sample has a very low expansion potential (EI = 9).

Atterberg Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) were determined for representative samples of the fine grained on-site soils 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

Settlement, collapse, and swell predictions under anticipated loads were made on the basis of a one-dimensional consolidation tests. The tests were performed in general conformance with Test Method ASTM D 2435 and ASTM D 5333. Loads were applied in a geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The results of the consolidation test are graphically presented in the figures in Appendix A.

Direct Shear

The angle of internal friction and cohesion were determined for relatively undisturbed soil samples by means of a direct shear tests. These tests were performed in general accordance with Test Method ASTM D 3080. Three specimens were prepared for each test. The test specimens were saturated and then sheared under normal loads at a constant rate of strain. Results are presented on the figures in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

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

Parameter	**B-1,3,4, and 5 0 to 5 feet**
pH	7.31
Chloride	91 ppm
Sulfate	0.0072%
Resistivity	2,300 ohm-cm

The chloride content of near-surface soils was determined for a selected sample in accordance with California Test Method No. 422. The results of this test indicate that the on-site soils have a low exposure to chloride. The results of testing of soil pH and resistivity were determined in accordance with California Test Method No. 643.

These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The test results on a near surface bulk sample from the site generally indicate that the tested on-site soils have moderate corrosive potential when in contact with ferrous materials. We recommend that a corrosion engineer review these results in order to provide specific corrosion protection as well as other protection for other buried metal materials.

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample in accordance with California Test Method No. 417. Our laboratory test data indicated that near surface soils contain approximately 0.0072 percent of water soluble sulfates. Based on the 2016 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-11, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-11 a negligible exposure to sulfate can be expected for concrete placed in contact with the tested on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

Photoionization Detector (PID) Screening

Soil samples taken from our subsurface exploration were screened with a Photoionization Detector (PID) to check for the possible presence of volatile vapors. No volatile vapors were detected during the screening of soil samples collected from any of the borings with a PID. Additionally, no odors were detected or stains observed that might suggest some form of contamination. PID field-screening results are included on the soil boring logs.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 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 groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2016 CBC and applicable local codes. Based upon the encountered subsurface soils, a Site Class D is recommended for design.

5.2 Seismic Hazard Zones

Our review of the published Seismic Hazard Evaluation Report for the Venice Quadrangle, within which the subject site is located, indicates that the subject site is within a designated Liquefaction Hazard Zone. A liquefaction analysis has therefore been performed.

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.

5.3 Landslide Hazards

The subject site does not lie within a mapped Landslide Hazard Zone based on our review of the published Seismic Hazard Evaluation Report for the Venice Quadrangle. Since the subject site is not located near unstable slope, mitigation of landslide hazards is not necessary for the site.

6.0 PRELIMINARY 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 preliminary conclusions and recommendations are incorporated in the design and project specifications.

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

According to the maps of known active fault near-source zones the Newport Inglewood and Santa Monica faults are the closest known active faults and are located about 3.2 and 4.5 miles from the site, respectively. These faults would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 6.3, as determined from deagredation analysis from the USGS website.

Within the 2016 CBC, the five-percent damped design spectral response accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are used to determine the seismic design base shear. These parameters, which are a function of the site's seismicity and soil, are also used as parts of triggers for other code requirements. The following values are determined by using the USGS published U.S. Seismic Design Maps program based upon the 2016 CBC referenced ASCE 7 (with July 2013 errata).

CBC 2016, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.3.1(1) for 0.2 second)	1.770
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.3.1(2) for 1.0 second)	0.648
Site Coefficient, F_a (Table 1613.3.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.3.3 (2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.770
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.972
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	1.180
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.648

Liquefaction

The Seismic Hazard Zone Report for the Venice 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 2016 CBC, California Geological Survey (CGS) Special Publication 117A, and additional City of Los Angeles (CLA) requirements for liquefaction analysis. General requirements by the CLA in performing liquefaction analyses are outlined in the *City of Los Angeles, July 16, 2014 Memorandum for 2014 Los Angeles Building Code (LABC) Requirements* and include the requirement of ground motions to be determined in accordance with the 2014 LABC. For the 475- and 2,475-year return periods the memorandum specifies the use of ⅔ PGA_M, and PGA_M and a factor of safety of 1.1, or 1.0, respectively, in the determination of liquefiable soils. Further, a fine-grained soil is considered non-liquefiable if the soil has a PI > 18 and test results indicate the soil is insensitive; or PI < 12 with moisture content > 85% of the liquid limit.

For this analysis we used the soil profiles identified with Borings B-1 and B-4, the historic high groundwater depth of 7 feet, and site accelerations (PGA_M and ⅔ PGA_M) of 0.64g and 0.43g, as obtained from the USGS web site and determined from ASCE-07. A corresponding site moment magnitudes of 6.3 was determined using 2008 Deaggregation methods published by USGS. As the actual historic high groundwater table is at a greater elevation than the current groundwater elevation encountered during our subsurface exploration, saturated moisture contents of the soil with a PI < 12 above the current water table were calculated and used in our analysis. 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 @ 6 ft.	52	27*	30	0.58
B-1 @ 10ft.	52	27*	32	0.62
B-1 @ 15ft.	52	30*	30	0.58
B-1 @ 20 ft.	52	30*	33	0.63
B-1 @ 25 ft.	34	5	29	0.85
B-1 @ 35 ft.	27	3	21	0.78
B-1 @ 40 ft.	30	3	17	0.57
B-1 @ 45 ft.	44	20	29	0.66
B-4 @ 6 ft.	38	8	29**	0.68
B-4 @ 15ft.	56	28*	34	0.61
B-4 @ 20 ft.	31	6	22	0.71
B-4 @ 25 ft.	30	4	28	0.93

B-4 @ 30 ft.	41	18*	32	0.78
B-4 @ 35 ft.	41	18*	31	0.76
B-4 @ 40 ft.	30	3	29	0.97
B-4 @ 45 ft.	29	7	22	0.76

* Soil tested and considered to be non-sensitive with Sensitivity ranging from 1.1 to 1.4.

** The soil moisture content represents the calculated saturated moisture content of the soil.

The results of our analysis with a high water table of 7 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	1.5	1.6
B-4	2.9	2.9

The total and differential seismic and static settlements for the borings were added together resulting in the following ranges:

Settlement Type	Settlement (inches)
Total Seismic	1.5 to 2.9
Total Static	0.5 inch
Differential Seismic	1.4 (2.9 minus 1.5)
Differential Static	0.25
Total Seismic + Static	
Total	**2.0 to 3.4**
Differential	**1.6**

The Los Angeles Building Code (LABC) requires a mat/slab foundation when the seismic plus static total settlements are greater than 2 inches, but less than 4 inches, and when the seismic plus static total differential settlements are greater than ¾ inch and less than 2 inches. A mat/slab foundation system is recommended in accordance with the Los Angeles Building Code (LABC) and foundation requirements based upon total combined seismic plus static settlements of 2.0 to 3.4 inches, and total combined seismic plus static differential settlement of 1.6 inches.

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.

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. Due to absence of any slope or channel within or near the site, the potential for lateral spreading in our opinion is considered to be low.

Liquefaction–Induced Potential for Surface Manifestation

Since the site soils are not susceptible to near surface (within the upper 25 feet) or significant soil liquefaction, surface manifestations resulting from soil liquefaction at this site are considered to be low.

6.2 Preliminary Site Development Recommendations

The following recommendations for future site development have been based upon the proposed 3-story structure, potential foundation bearing grades, the conditions encountered at the test boring locations and the time of year in which the exploration was performed.

Site Clearing and Demolition

Clearing operations should include the removal of all existing vegetation and debris within the area of the future new site development. Trees and large shrubs to be removed should be grubbed out to include their stumps and major root systems. All soils disturbed during site clearing should also be removed to expose suitable bearing soils. Some of the near surface fine grained soils have in-situ moisture contents that were very moist and above the optimum moisture content to an extent that may create difficulties in achieving the required compaction. Therefore, air-drying or import soil may be required.

Should any unusual soil conditions or subsurface structures be encountered 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 building areas. Utilities that are not to be reused should be capped off 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.

Building Area Subgrade Preparation

Due to the presence of undocumented fill and possible fill and the variable strength characteristics of the near surface on-site soils, some grading and over-excavation will be required for development of new structures. Over-excavation is anticipated to extend to maximum depths of about 5 feet below existing ground surfaces, and at least 2 feet below the bottom of foundations, whichever is greater. The soils exposed at the base of this recommended over-excavation should be examined by the geotechnical engineer to document that all undocumented fill and other unsuitable soils have been removed. Prior to placement of fill, the exposed surfaces approved for fill placement should first be scarified to an approximate depth of at least 6 inches, moisture conditioned and then recompacted in place to a minimum relative compaction of at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00). A representative of the project geotechnical consultant should be present on site during grading operations to verify proper placement and adequate compaction of all fills.

The preliminary estimated overexcavation of the existing fill and possible fill is considered the maximum required amount of removal of undocumented fill per City of Los Angeles requirements. The depth of required removal should be confirmed with additional field testing including shallow borings and test pits.

Upon receipt of detailed building plans, building specific recommendations will be provided by our firm upon request.

Proofroll and Compact Subgrade

The subgrades within future pavement area improvements 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 subgrade should be scarified to a minimum depth of 8 inches, moisture conditioned and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557-00) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable low expansive (EI < 51) structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

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

Reuse of On-site Soil

On-site low expansive material (EI < 51) may be reused as structural compacted fill within future parking lot and building areas provided they do not contain oversized materials and significant quantities of organic matter or other deleterious materials. 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. Some drying of the site soils may be necessary prior to their use as engineered fill, based on the in-situ moisture contents of these soils.

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. The site should be graded to prevent water from ponding within construction areas and/or flowing into excavations. Accumulated water must be removed immediately along with any unstable soil. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing grade.

The degree of subgrade instability and associated remedial construction is dependent, in part, upon precautions taken by the contractor to protect the subgrade during site development.

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

Fill Placement

Material for engineered fill should be select 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 and pavement areas.

All on-site fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned and then compacted in place to at least 90 percent of the Modified Proctor maximum density. The upper 1 foot of the pavement subgrade should have a minimum in-place density of at least 95%. 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

The soils imported to the site for use as structural fill should consist of low expansive soils (EI < 51). 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. Shrinkage will be a function of actual materials to be used during grading and construction.

6.3 Preliminary Construction Considerations

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

Construction Dewatering

As mentioned previously, groundwater was encountered at 20 to 25 feet during our subsurface investigation. Therefore, groundwater is not expected to impact shallow 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.

Soil Excavation

Some slope stability problems should be expected in steep, unbraced excavations considering the granular 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.

6.4 Preliminary Foundation Recommendations

A mat/slab foundation is considered suitable with a minimum 2 foot structural fill layer, allowable bearing capacity in the range of 2,000 to 3,000 psf, and a modulus of subgrade reaction (k_s) in the range of 50 to 70 pci. The recommended allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Reinforcing

The design of the foundations and the determination of the steel reinforcing should be performed by a qualified structural engineer.

Lateral Load Resistance

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

Assuming the foundation subgrade consists of a silty sand to sandy silt, a coefficient of friction of 0.30 may be used with dead load forces for foundations placed on newly placed compacted fill soil. A higher coefficient is possible with a higher quality granular fill backfill. An allowable passive earth pressure of 250 psf per foot of foundation depth (pcf) below the lowest adjacent grade may be used for the sides of foundations placed against newly placed structural fill. The maximum recommended allowable passive pressure is 1,500 psf.

Bearing Material Criteria

Soil suitable to serve as the structural fill subgrade should exhibit an average N value for non-cohesive soils and unconfined compressive strength for cohesive soils based on the recommended allowable soil bearing pressure. For design and construction estimating purposes, suitable bearing soils are expected to be encountered at the recommended over-excavation depths with structural fill below the foundation and mat/slab. However, field testing by the geotechnical engineer within the foundation bearing soils is recommended to document that the foundation support soils possess the minimum strength parameters noted above. Testing may consist of Dynamic Cone Penetration tests (per ASTM Special Publication 399), pocket penetrometer tests, or other tests as deemed suitable by the Geotechnical Engineer. If unsuitable bearing soils are encountered, they should be recompacted in-place, if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade.

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 foundations may be supported at nominal depth below the floor. All foundations must be protected against weather and water damage during and after construction, and must be supported by the minimum recommended structural fill layer.

6.5 Mat/Slab Recommendations

Subgrade

The mat/slab 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-slab 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/slab based on the recommendations presented in the foundation section of this report.

The mat/slab 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 retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture on future flooring materials. The vapor retarder 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 or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Where the foundations or mat/slab are underlain directly by the base course followed by the vapor barrier additional reinforcing cover is not considered necessary.

Estimated Settlement

Post-construction total and differential movements of the mat/slab designed and constructed in accordance with the recommendations provided in this report are estimated to be on the order of those estimated for foundations. The mat/slab differential movement is estimated to be in the range of ½ to ¼ inch over 40 feet depending on the thickness of the structural fill layer and/or the use of a geo grid.

Therefore, the total seismic plus static settlement is 2.0 to 3.4 inches, and the total differential seismic plus static is 1.6 inches.

6.6 Pavement Recommendations

New Pavement Subgrades

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 5 to 20. An R-value of 5 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	7	10	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.

6.7 Recommended Construction Materials Testing Services

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

6.8 Basis of Report

This report is based on Giles' proposal, dated June 6, 2018, and is referenced by Giles' proposal number 2GEP-1806004. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

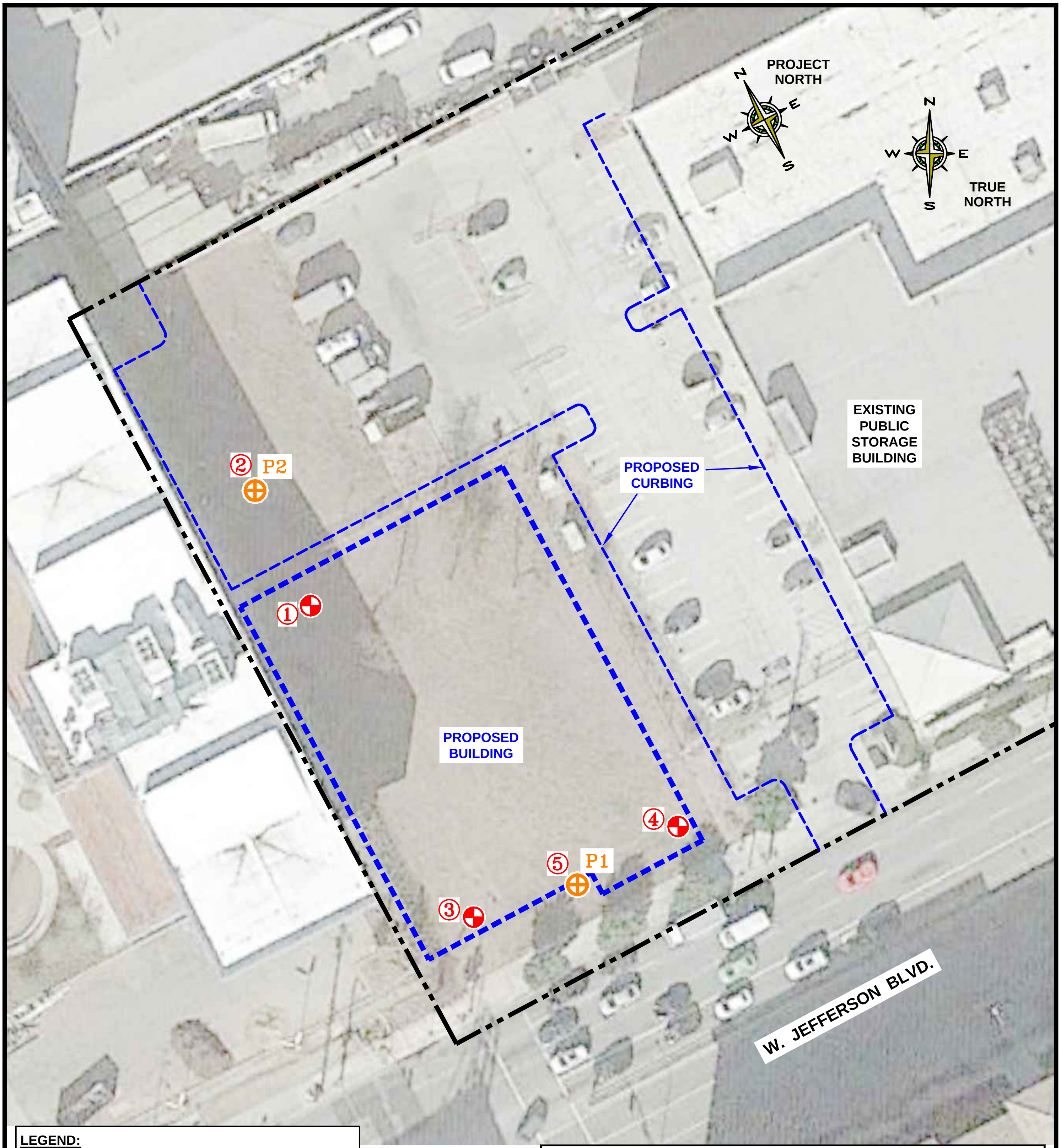
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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



LEGEND:

① 🟥 GEOTECHNICAL TEST BORING

⑤ P1 ⊕ GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING

0 25' 50'

APPROXIMATE SCALE

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "OPTION A", DATED 4-24-18, PREPARED BY JOSHUA HUNT.

GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817
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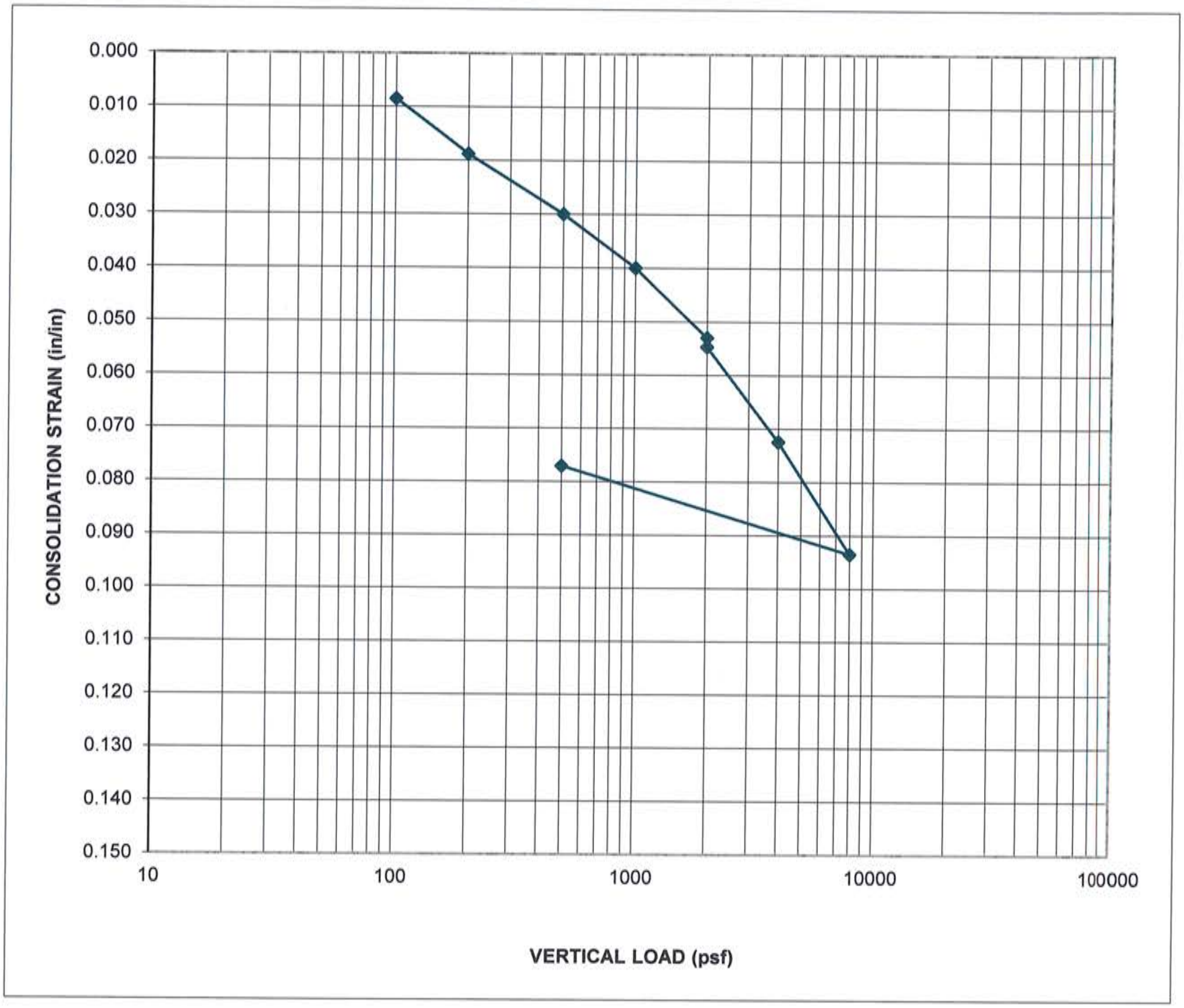
FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED 3-STORY BUILDING
PUBLIC STORAGE
12681 W. JEFFERSON BOULEVARD
LOS ANGELES (PLAYA VISTA), CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
JLM		approx. 1"=50'	08-31-18	--

PROJECT NO.: 2G-1806003

CAD No. 2g1806003-blp

CONSOLIDATION / SWELL TEST ASTM D2435/ASTM D4546



Classification	Silt, trace fine Sand		
Boring No.	B-3		
Sample No.	3-CS	Initial Moisture Content (%)	24.5
Depth (ft.)	6 to 7.5	Final Moisture Content (%)	24.8
Elevation		Natural Density (pcf)	119.2
Liquid Limit		Initial Dry Density (pcf)	95.7
Plastic Limit		Final Dry Density (pcf)	104.8
Specimen Diameter (in.)	2.42	Collapse @ 2,000 psf	0.1%
Initial Specimen Thickness (in.)	1.00		

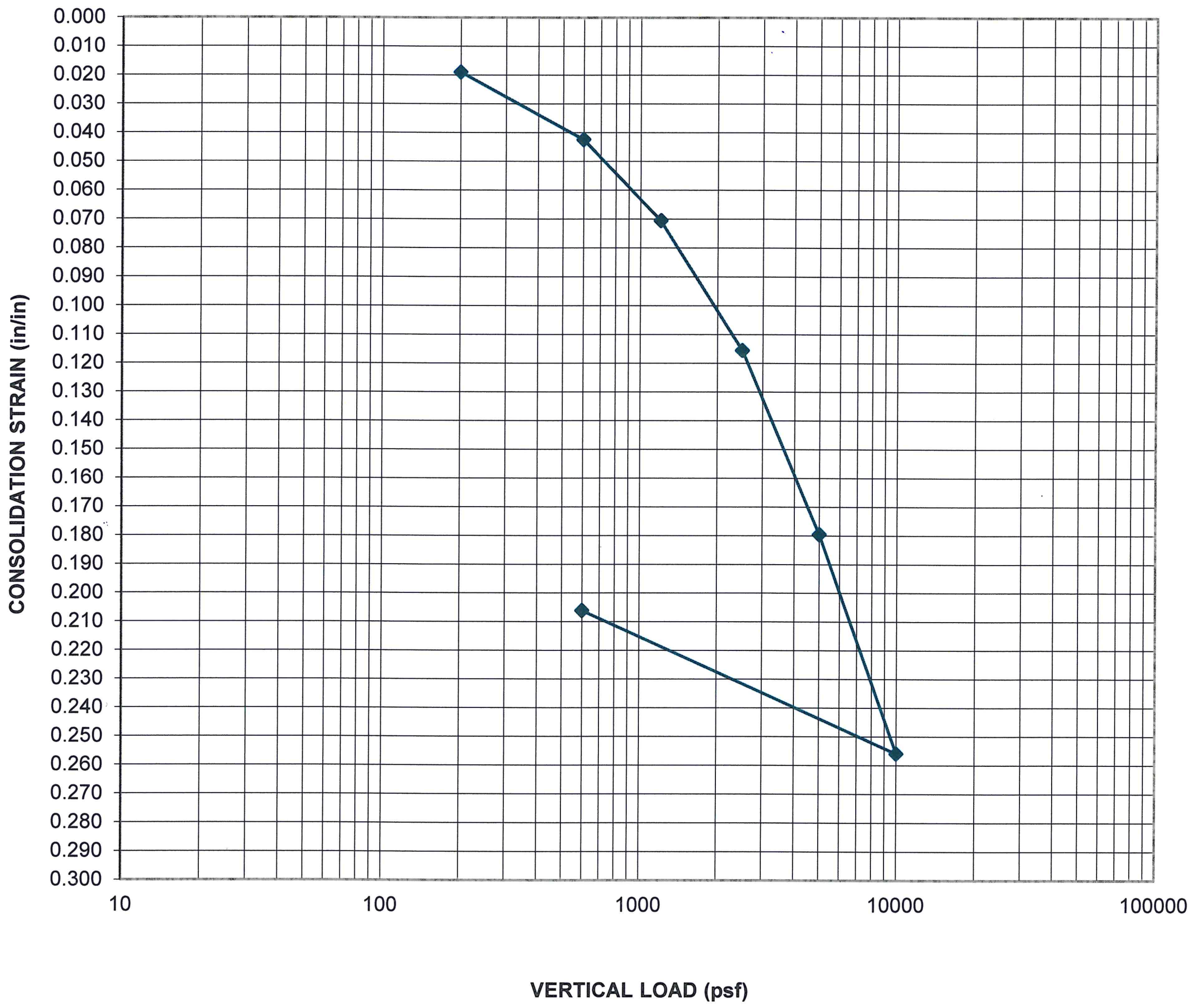
Sample inundated at 2,000 psf pressure

Project:	PS Playa Vista
Client:	Public Storage
Project No.:	2G-1806003
Figure No.:	2

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
1965 NORTH MAIN STREET, ORANGE, CALIFORNIA
OFFICE: 714-279-0817 FAX : 714-279-9687

CONSOLIDATION / SWELL TEST ASTM D2435/ASTM D4546



Classification	Silty Clay
Boring No.	B-3
Sample No.	6-ST
Depth (ft.)	16 to 18.5
Elevation	-1.5ft
Liquid Limit	
Plastic Limit	
Specimen Diameter (in.)	2.90
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	31.9
Final Moisture Content (%)	24.3
Natural Density (pcf)	111.0
Initial Dry Density (pcf)	84.1
Final Dry Density (pcf)	92.1
Swell/Collapse @ 2500 psf	0

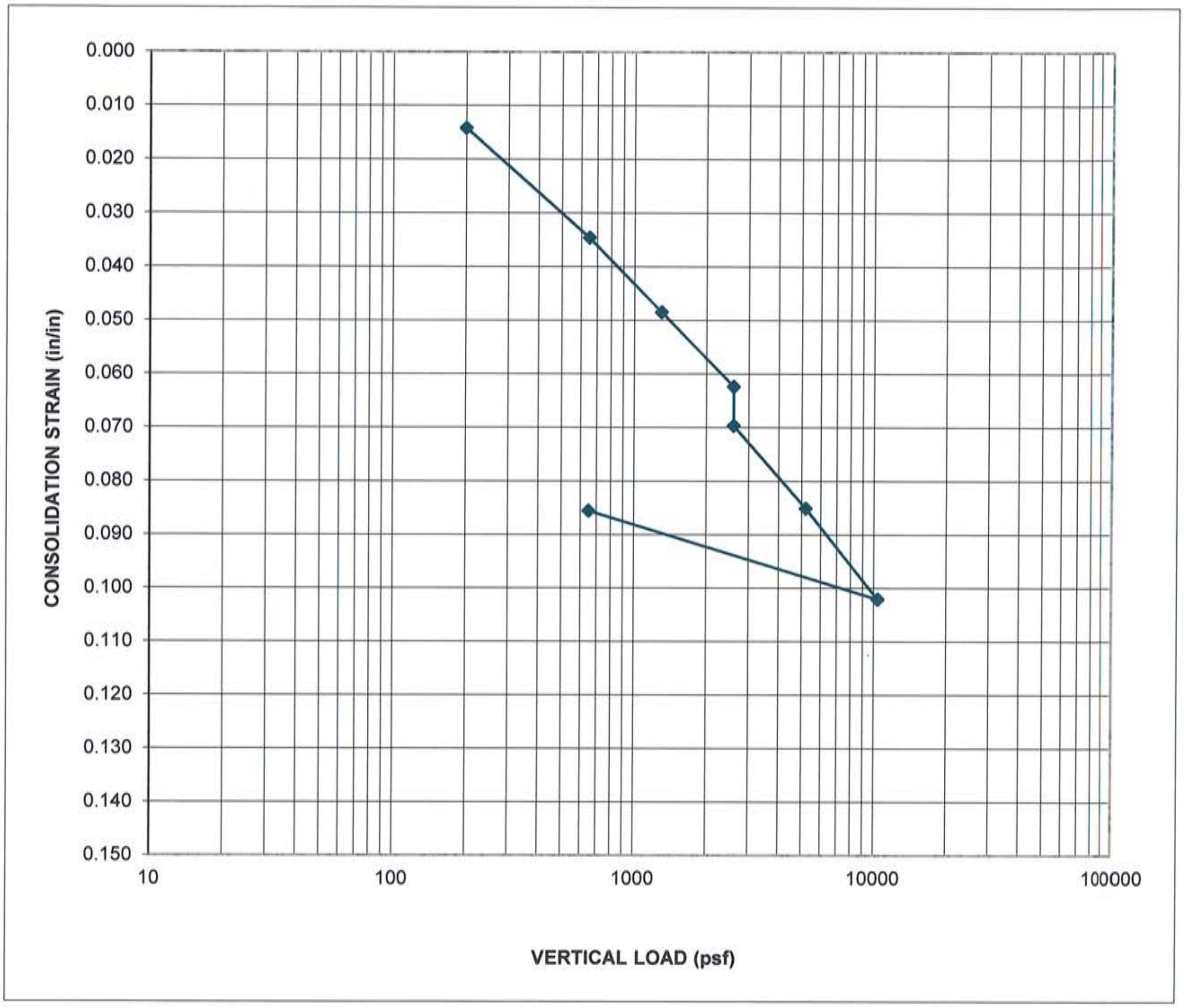
Sample inundated at 2,500 psf pressure

Project:	PS Playa Vista
Client:	Public Storage
Project No.:	2G-1806003
Figure No.:	3

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
1965 NORTH MAIN STREET, ORANGE, CALIFORNIA
OFFICE: 714-279-0817 FAX : 714-279-9687

CONSOLIDATION / SWELL TEST ASTM D2435/ASTM D4546



Classification	Fine Sand, some Silt		
Boring No.	B-4		
Sample No.	5-ST	Initial Moisture Content (%)	23.5
Depth (ft.)	11.5 to 14	Final Moisture Content (%)	27.1
Elevation	4 ft	Natural Density (pcf)	115.4
Liquid Limit		Initial Dry Density (pcf)	93.5
Plastic Limit		Final Dry Density (pcf)	102.3
Specimen Diameter (in.)	2.90	Collapse @ 2600 psf	0.7%
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 2600 psf pressure

Project:	PS Playa Vista
Client:	Public Storage
Project No.:	2G-1806003
Figure No.:	4

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-
1965 NORTH MAIN STREET, ORANGE, CALIFORNIA
OFFICE: 714-279-0817 FAX : 714-279-9687

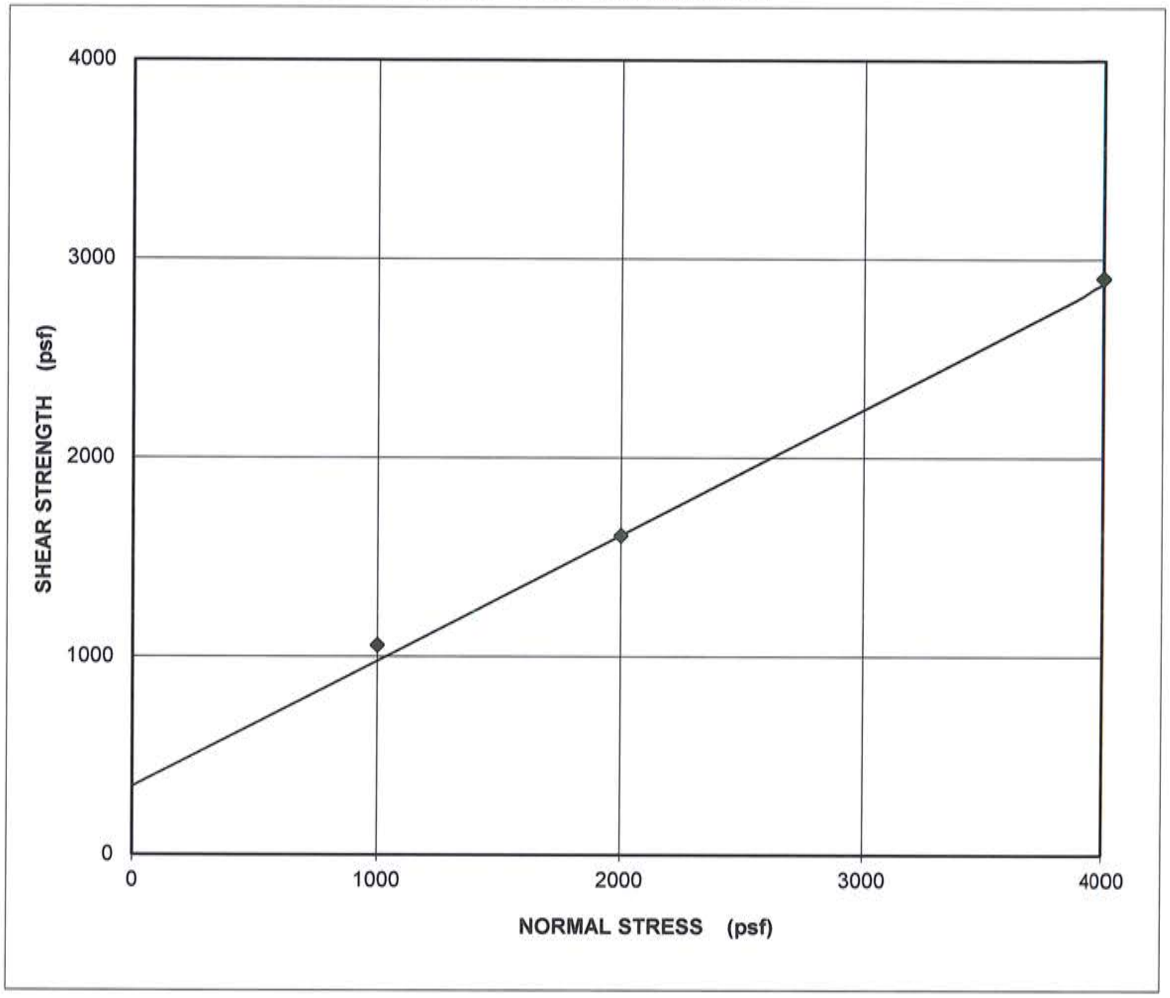
Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865

OFFICE: 714-279-0817 FAX: 714-279-9687

DIRECT SHEAR TEST (ASTM D3080)



Classification Fine Sand, some Silt

Boring No. B-4 Depth 11.5 to 14.0 feet (El. 4.0 ft)

		Initial Specimen Properties:	
		Diameter (in.)	2.4
		Height (in.)	1.0
PROJECT:	Public Storage - Los Angeles	Moisture Content (%)	24
		Remolded Density (pcf)	
CLIENT:	Public Storage	Dry Density (pcf)	93.5
		LL	
PROJECT NO.:	2G-1806003	PL	NP
DATE:	8/9/2018	C (psf)	300
FIGURE	5	PHI (degrees)	32

BORING NO. & LOCATION: B-1	# TEST BORING LOG
SURFACE ELEVATION: 20 feet	PROPOSED PUBLIC STORAGE 3-STORY BUILDING
COMPLETION DATE: 07/10/18	12681 W. JEFFERSON BOULEVARD LOS ANGELES, CA
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1806003

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
Brown Clayey fine Sand - Damp (Fill)			1-SS	10				13	BDL	EI=9 Very low composite sample
Brown Silty fine Sand - Moist (Fill to Possible Fill)			2-SS	12				9	BDL	
Gray Silty Clay - Very Moist			3-SS	7		2.5		30	BDL	LL=52 PI=27
	10	10	4-SS	9		2.75		32	BDL	Dd=90.7 pcf
Dark Brown Silty Clay - Very Moist			5-SS	5		2.5		29	BDL	LL=52 PI=30 Dd=93.2 pcf
	20	0	6-SS	6		2.5		32	BDL	Dd=92.8 pcf
Dark Gray Silt, some fine Sand - Very Moist	▽		7-SS	8				29	BDL	LL=34 PI=5 P_{200}=78%
Brown fine to coarse Sand, little Silt - Wet	30	-10	8-SS	25				14	BDL	P_{200}=11%
Dark Gray fine Sandy Silt - Very Moist			9-SS	15				21	BDL	LL=27 PI=3 P_{200}=63%
Gray coarse Sand, little Silt - Wet	40	-20	10-SS	8				17	BDL	P_{200}=13%
Dark Gray Clay - Wet			11-SS	7		3.0		29	BDL	LL=44 PI=20 P_{200}=81%
Gray fine to coarse Sandy Silt - Very Moist	50	-30	12-SS	38				28	BDL	
Gray fine to coarse Sand - Wet			13-SS	22				29	14	
	60	-40	14-SS	50/6"					BDL	

Boring Terminated at about 66.5 feet (EL. -46.5')

	Water Observation Data
▽	Water Encountered During Drilling: 25 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

BDL = Below Detection Limit

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-2

SURFACE ELEVATION:
21 feet

COMPLETION DATE:
07/10/18

FIELD REP:
TREVOR SLAZAS

TEST BORING LOG

PROPOSED PUBLIC STORAGE 3-STORY BUILDING

12681 W. JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-1806003

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
Brown Silty fine Sand, trace Gravel - Moist to Very Moist (Fill to Possible Fill)		20.0	1-SS	12				7	BDL	
	2.5	17.5	2-SS	18				10	BDL	P_{200}=41%
Boring Terminated at about 5 feet (EL. 16')	5.0									

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

BDL = Below Detection Limit

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: 17 feet	PROPOSED PUBLIC STORAGE 3-STORY BUILDING 12681 W. JEFFERSON BOULEVARD LOS ANGELES, CA PROJECT NO: 2G-1806003	
COMPLETION DATE: 07/10/18		
FIELD REP: TREVOR SLAZAS		

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Brown Sandy Silt - Damp (Fill)		15	1-SS	12				8	BDL	
Brown Silty Sand, trace Gravel - Very Moist	5		2-CS	22				19	BDL	Dd=96.4 pcf
Light Brown Silt, trace fine Sand - Very Moist		10	3-CS	18				24	BDL	Dd=95.7 pcf
Brown Silty fine Sand - Very Moist	10	5	4-SS	8				30	BDL	
Black Silty Clay - Very Moist	15	0	5-SS	3				32	BDL	
			6-ST					32	BDL	Dd=84.1 pcf
Brown Silt, trace Clay and fine Sand - Very Moist	20	-5	7-SS	5				30	BDL	
Brown Silty fine Sand - Very Moist	▽ 25		8-CS	12				30	BDL	Dd=89.5 pcf

Boring Terminated at about 26.5 feet (EL. -9.5')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 25 ft.	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
░	Cave Depth At End of Drilling:	BDL = Below Detection Limit
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

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:
18 feet

COMPLETION DATE:
07/10/18

FIELD REP:
TREVOR SLAZAS

TEST BORING LOG

PROPOSED PUBLIC STORAGE 3-STORY BUILDING

12681 W. JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-1806003

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
Brown Clayey fine Sand - Moist (Fill)			1-SS	14		4.5+		13	BDL	
Brown Sandy Clay - Moist (Fill)			2-SS	7				13	BDL	
Light Brown fine Sandy Silt - Moist		10	3-SS	6				20	BDL	Dd=93.3 pcf P_{200}=69% LL=38 PI=8
Brown fine Sand, some Silt - Very Moist	10		4-SS	10				26	BDL	P_{200}=34%
			5-ST					24	BDL	Dd=93.5 pcf
Dark Brown Clay - Very Moist		0	6-SS	5		2.25		34	BDL	LL=56 PI=28 Dd=94.5 pcf
Dark Gray Clayey Silt, little fine Sand - Very Moist	▽ 20		7-SS	6				22	BDL	LL=31 PI=6 P_{200}=84% Dd=102.1 pcf
Gray to Reddish Brown Sandy Silt - Very Moist		-10	8-SS	7				28	BDL	P_{200}=54% LL=30 PI=4
Gray fine Sandy Clay - Very Moist	30		9-SS	3		1.5		32	BDL	P_{200}=81%
		-20	10-SS	4		1.5		31	BDL	LL=41 PI=18
Dark Gray Silt, some fine Sand - Wet	40		11-SS	11				29	BDL	LL=30 PI=3 P_{200}=75%
Dark Gray fine Sandy Clay - Wet		-30	12-SS	6				22	BDL	P_{200}=53% LL=29 PI=7
Gray fine to coarse Sand, some Gravel - Wet	50		13-SS	29				8	BDL	
No recovery		-40	14-SS	41					BDL	
Gray fine to coarse Sand - Wet	60		15-SS	42				13	BDL	

Boring Terminated at about 66.5 feet (EL. -48.5')

	Water Observation Data
▽	Water Encountered During Drilling: 20 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

BDL = Below Detection Limit

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

BORING NO. & LOCATION:
B-5/P-1

SURFACE ELEVATION:
17 feet

COMPLETION DATE:
07/10/18

FIELD REP:
TREVOR SLAZAS

TEST BORING LOG

PROPOSED PUBLIC STORAGE 3-STORY BUILDING

12681 W. JEFFERSON BOULEVARD
LOS ANGELES, CA

PROJECT NO: 2G-1806003

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
Brown Silt, some fine Sand and Gravel - Moist (Fill)										
		15.0	1-SS	10				10	BDL	
	2.5									
		12.5	2-SS	6				15	BDL	P_{200}=67%
Boring Terminated at about 5 feet (EL. 12')	5.0									

	Water Observation Data
▽	Water Encountered During Drilling: None
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

BDL = Below Detection Limit

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

PS LA Jefferson Blvd, Playa Vista,

Hole No.=B1 ***Water Depth=7 ft***

Magnitude=6.3
Acceleration=0.43g

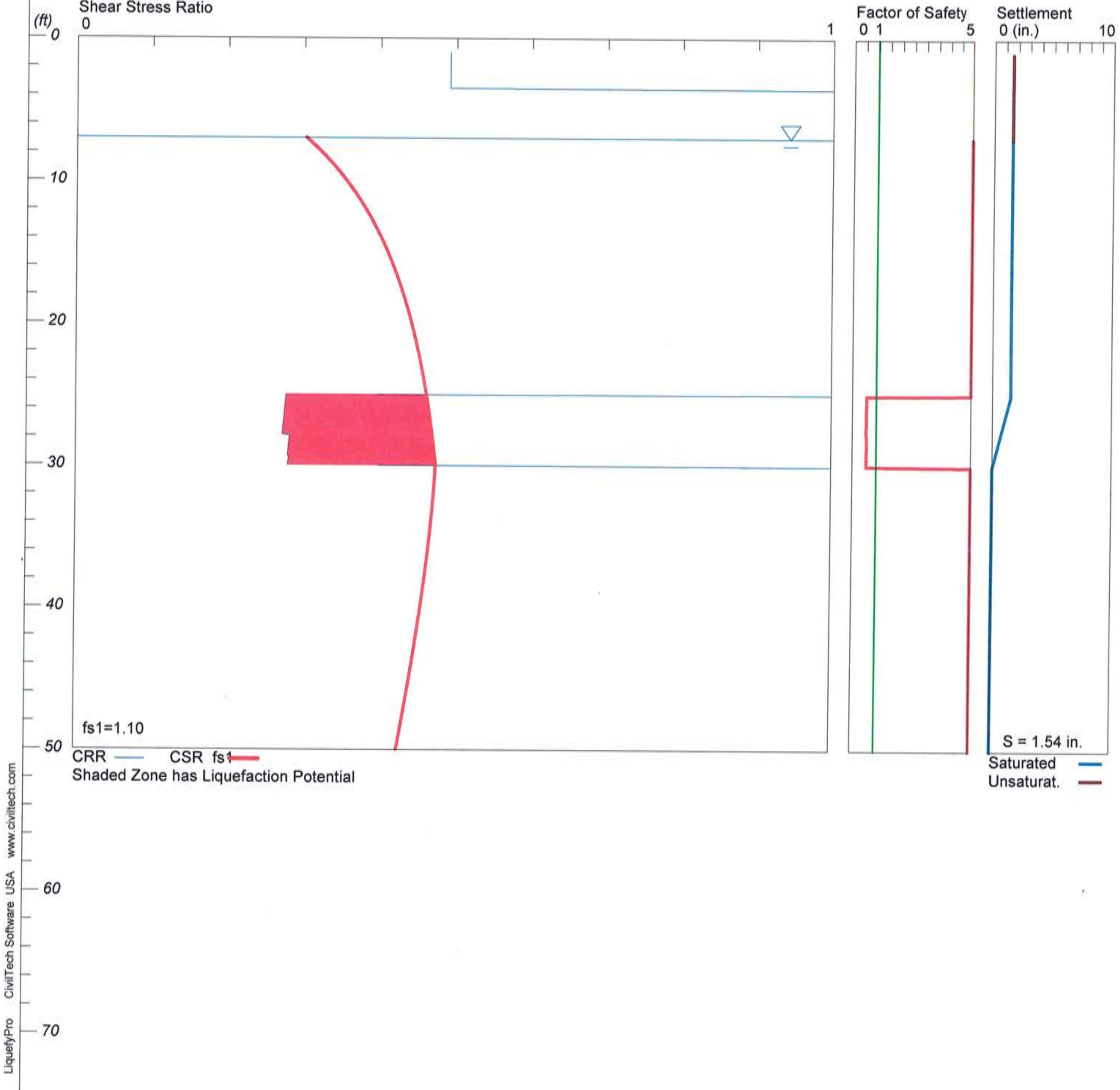


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Plate A-1

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                                            LIQUEFACTION ANALYSIS SUMMARY

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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   9/10/2018      4:35:32 PM

        Input File Name: P:\John Maier\Public Storage\Geo\LA 3 - Playa
Vista\B-1   8-3-18  10 in 50.liq
        Title:  PS LA Jefferson Blvd, Playa Vista,
        Subtitle:

        Surface Elev.=
        Hole No.=B1
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration= 0.43 g
        Earthquake Magnitude= 6.30

 Input Data:
        Surface Elev.=
        Hole No.=B1
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration=0.43 g
        Earthquake Magnitude=6.30
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        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: No
        * Recommended Options

        In-Situ Test Data:
        Depth    SPT     gamma    Fines
        ft               pcf      %
        ____________________________________
        1.00     10.00   120.00   30.00
        3.50     12.00   120.00   30.00
        6.00     7.00    120.00   NoLiq
        10.00    9.00    120.00   NoLiq
        15.00    5.00    120.00   NoLiq
        20.00    6.00    120.00   NoLiq
        25.00    8.00    120.00   78.00
        30.00    25.00   120.00   11.00
        35.00    15.00   120.00   NoLiq
        40.00    8.00    120.00   NoLiq
        48.50    7.00    120.00   NoLiq
        ____________________________________
```

```
Output Results:
        Settlement of Saturated Sands=1.53 in.
        Settlement of Unsaturated Sands=0.01 in.
        Total Settlement of Saturated and Unsaturated Sands=1.54 in.
        Differential Settlement=0.772 to 1.019 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        ___________________________________________________
        1.00    0.49    0.31    5.00    1.53    0.01    1.54
        2.00    0.49    0.31    5.00    1.53    0.01    1.54
        3.00    0.49    0.31    5.00    1.53    0.01    1.54
        4.00    3.12    0.30    5.00    1.53    0.00    1.54
        5.00    3.12    0.30    5.00    1.53    0.00    1.54
        6.00    2.00    0.30    5.00    1.53    0.00    1.53
        7.00    2.00    0.30    5.00    1.53    0.00    1.53
        8.00    2.00    0.32    5.00    1.53    0.00    1.53
        9.00    2.00    0.34    5.00    1.53    0.00    1.53
        10.00   2.00    0.36    5.00    1.53    0.00    1.53
        11.00   2.00    0.37    5.00    1.53    0.00    1.53
        12.00   2.00    0.38    5.00    1.53    0.00    1.53
        13.00   2.00    0.39    5.00    1.53    0.00    1.53
        14.00   2.00    0.40    5.00    1.53    0.00    1.53
        15.00   2.00    0.41    5.00    1.53    0.00    1.53
        16.00   2.00    0.42    5.00    1.53    0.00    1.53
        17.00   2.00    0.43    5.00    1.53    0.00    1.53
        18.00   2.00    0.43    5.00    1.53    0.00    1.53
        19.00   2.00    0.44    5.00    1.53    0.00    1.53
        20.00   2.00    0.44    5.00    1.53    0.00    1.53
        21.00   2.00    0.45    5.00    1.53    0.00    1.53
        22.00   2.00    0.45    5.00    1.53    0.00    1.53
        23.00   2.00    0.46    5.00    1.53    0.00    1.53
        24.00   2.00    0.46    5.00    1.53    0.00    1.53
        25.00   2.00    0.46    5.00    1.53    0.00    1.53
        26.00   0.28    0.47    0.59*   1.24    0.00    1.24
        27.00   0.27    0.47    0.59*   0.93    0.00    0.93
        28.00   0.28    0.47    0.60*   0.62    0.00    0.62
        29.00   0.28    0.47    0.59*   0.32    0.00    0.32
        30.00   0.28    0.48    0.59*   0.02    0.00    0.02
        31.00   3.13    0.47    5.00    0.00    0.00    0.00
        32.00   3.12    0.47    5.00    0.00    0.00    0.00
        33.00   3.11    0.47    5.00    0.00    0.00    0.00
        34.00   3.10    0.47    5.00    0.00    0.00    0.00
        35.00   3.09    0.47    5.00    0.00    0.00    0.00
        36.00   2.00    0.47    5.00    0.00    0.00    0.00
        37.00   2.00    0.46    5.00    0.00    0.00    0.00
        38.00   2.00    0.46    5.00    0.00    0.00    0.00
        39.00   2.00    0.46    5.00    0.00    0.00    0.00
        40.00   2.00    0.46    5.00    0.00    0.00    0.00
        41.00   2.00    0.45    5.00    0.00    0.00    0.00
        42.00   2.00    0.45    5.00    0.00    0.00    0.00
        43.00   2.00    0.45    5.00    0.00    0.00    0.00
        44.00   2.00    0.45    5.00    0.00    0.00    0.00
        45.00   2.00    0.44    5.00    0.00    0.00    0.00
        46.00   2.00    0.44    5.00    0.00    0.00    0.00
        47.00   2.00    0.44    5.00    0.00    0.00    0.00
        48.00   2.00    0.43    5.00    0.00    0.00    0.00
        49.00   2.00    0.43    5.00    0.00    0.00    0.00
        50.00   2.00    0.43    5.00    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:  Welcome to LiquefyPro!
```

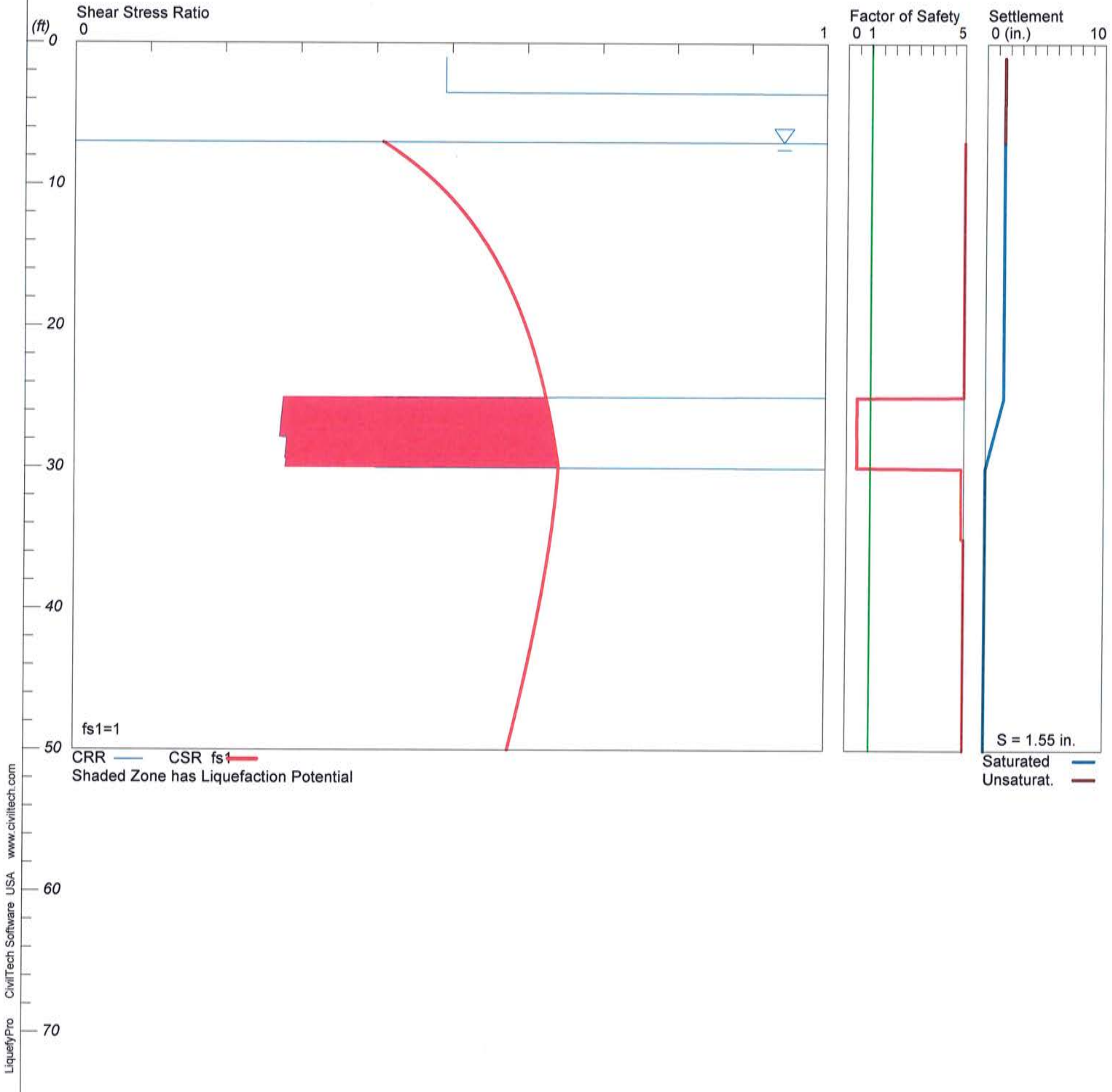
```
______
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm                Cyclic resistance ratio from soils
        CSRsf               Cyclic stress ratio induced by a given earthquake
(with user request factor of safety)
        F.S.                Factor of Safety against liquefaction, 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

PS LA Jefferson Blvd, Playa Vista,

Hole No.=B1 Water Depth=7 ft

Magnitude=6.3
Acceleration=0.64g



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        Input File Name: P:\John Maier\Public Storage\Geo\LA 3 - Playa
Vista\B-1   8-3-18  2 in 50.liq
        Title:  PS LA Jefferson Blvd, Playa Vista,
        Subtitle:

        Surface Elev.=
        Hole No.=B1
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration= 0.64 g
        Earthquake Magnitude= 6.30

 Input Data:
        Surface Elev.=
        Hole No.=B1
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration=0.64 g
        Earthquake Magnitude=6.30
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        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=1)
        10. Use Curve Smoothing: No
        * Recommended Options

        In-Situ Test Data:
        Depth    SPT     gamma    Fines
        ft               pcf      %
        ____________________________________
        1.00     10.00   120.00   30.00
        3.50     12.00   120.00   30.00
        6.00     7.00    120.00   NoLiq
        10.00    9.00    120.00   NoLiq
        15.00    5.00    120.00   NoLiq
        20.00    6.00    120.00   NoLiq
        25.00    8.00    120.00   78.00
        30.00    25.00   120.00   11.00
        35.00    15.00   120.00   NoLiq
        40.00    8.00    120.00   NoLiq
        48.50    7.00    120.00   NoLiq
        ____________________________________
```

```
Output Results:
        Settlement of Saturated Sands=1.53 in.
        Settlement of Unsaturated Sands=0.02 in.
        Total Settlement of Saturated and Unsaturated Sands=1.55 in.
        Differential Settlement=0.775 to 1.023 in.

        Depth    CRRm     CSRfs    F.S.     S_sat.   S_dry    S_all
        ft                                  in.      in.      in.
        _______________________________________________________________
        1.00     0.49     0.42     5.00     1.53     0.02     1.55
        2.00     0.49     0.41     5.00     1.53     0.01     1.55
        3.00     0.49     0.41     5.00     1.53     0.01     1.54
        4.00     3.12     0.41     5.00     1.53     0.01     1.54
        5.00     3.12     0.41     5.00     1.53     0.00     1.54
        6.00     2.00     0.41     5.00     1.53     0.00     1.53
        7.00     2.00     0.41     5.00     1.53     0.00     1.53
        8.00     2.00     0.44     5.00     1.53     0.00     1.53
        9.00     2.00     0.46     5.00     1.53     0.00     1.53
        10.00    2.00     0.48     5.00     1.53     0.00     1.53
        11.00    2.00     0.50     5.00     1.53     0.00     1.53
        12.00    2.00     0.52     5.00     1.53     0.00     1.53
        13.00    2.00     0.53     5.00     1.53     0.00     1.53
        14.00    2.00     0.54     5.00     1.53     0.00     1.53
        15.00    2.00     0.56     5.00     1.53     0.00     1.53
        16.00    2.00     0.57     5.00     1.53     0.00     1.53
        17.00    2.00     0.58     5.00     1.53     0.00     1.53
        18.00    2.00     0.58     5.00     1.53     0.00     1.53
        19.00    2.00     0.59     5.00     1.53     0.00     1.53
        20.00    2.00     0.60     5.00     1.53     0.00     1.53
        21.00    2.00     0.61     5.00     1.53     0.00     1.53
        22.00    2.00     0.61     5.00     1.53     0.00     1.53
        23.00    2.00     0.62     5.00     1.53     0.00     1.53
        24.00    2.00     0.62     5.00     1.53     0.00     1.53
        25.00    2.00     0.63     5.00     1.53     0.00     1.53
        26.00    0.28     0.63     0.44*    1.24     0.00     1.24
        27.00    0.27     0.63     0.43*    0.93     0.00     0.93
        28.00    0.28     0.64     0.44*    0.62     0.00     0.62
        29.00    0.28     0.64     0.44*    0.32     0.00     0.32
        30.00    0.28     0.64     0.44*    0.02     0.00     0.02
        31.00    3.13     0.64     4.88     0.00     0.00     0.00
        32.00    3.12     0.64     4.88     0.00     0.00     0.00
        33.00    3.11     0.64     4.88     0.00     0.00     0.00
        34.00    3.10     0.64     4.88     0.00     0.00     0.00
        35.00    3.09     0.63     4.89     0.00     0.00     0.00
        36.00    2.00     0.63     5.00     0.00     0.00     0.00
        37.00    2.00     0.63     5.00     0.00     0.00     0.00
        38.00    2.00     0.62     5.00     0.00     0.00     0.00
        39.00    2.00     0.62     5.00     0.00     0.00     0.00
        40.00    2.00     0.62     5.00     0.00     0.00     0.00
        41.00    2.00     0.61     5.00     0.00     0.00     0.00
        42.00    2.00     0.61     5.00     0.00     0.00     0.00
        43.00    2.00     0.61     5.00     0.00     0.00     0.00
        44.00    2.00     0.60     5.00     0.00     0.00     0.00
        45.00    2.00     0.60     5.00     0.00     0.00     0.00
        46.00    2.00     0.59     5.00     0.00     0.00     0.00
        47.00    2.00     0.59     5.00     0.00     0.00     0.00
        48.00    2.00     0.59     5.00     0.00     0.00     0.00
        49.00    2.00     0.58     5.00     0.00     0.00     0.00
        50.00    2.00     0.58     5.00     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:  Welcome to LiquefyPro!
```

```
        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

PS LA Jefferson Blvd, Playa Vista,

Hole No.=B4   Water Depth=7 ft

Magnitude=6.3
Acceleration=0.43g

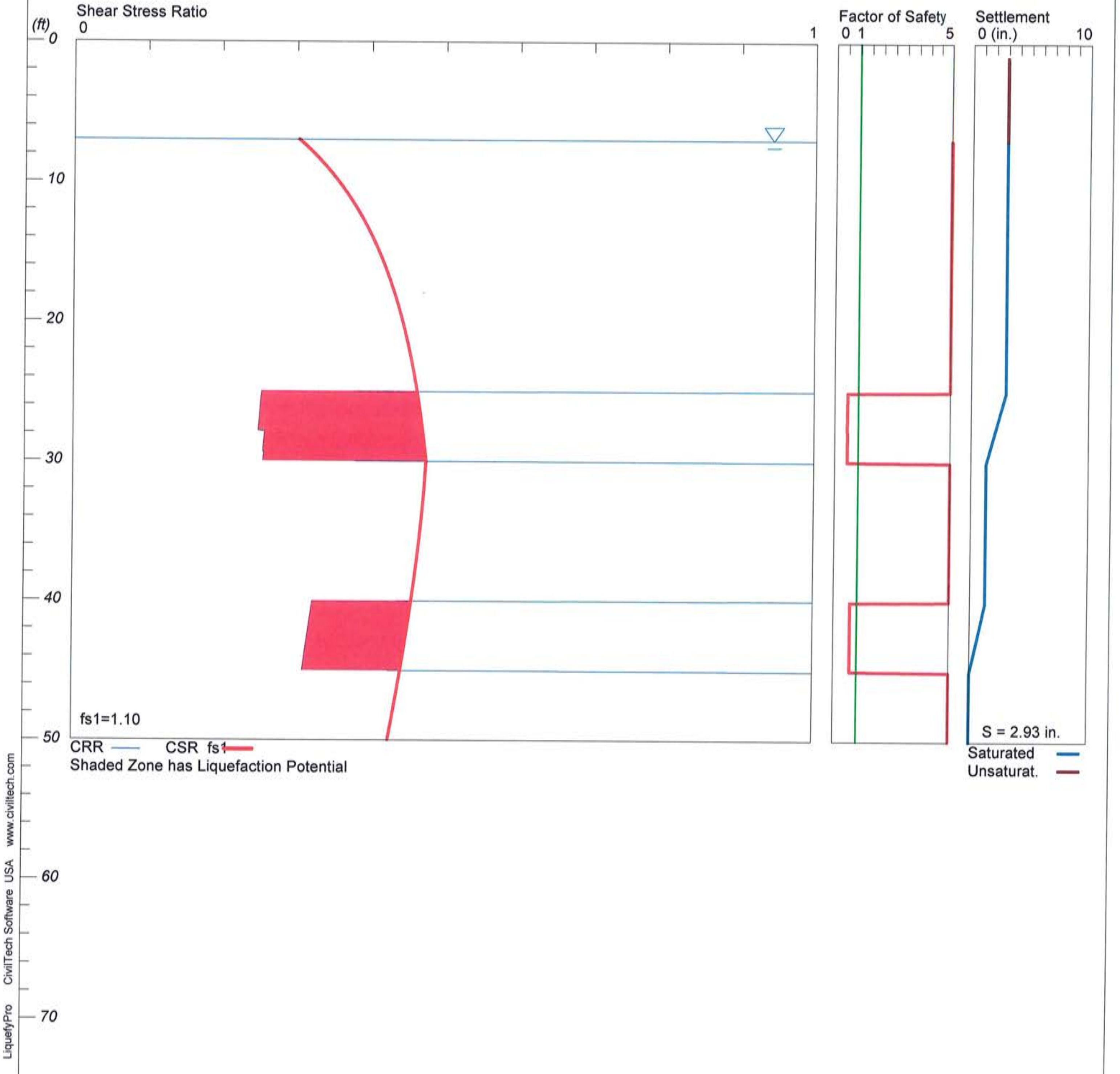

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Plate A-1

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                                                    LIQUEFACTION ANALYSIS SUMMARY

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        Input File Name: P:\John Maier\Public Storage\Geo\LA 3 - Playa
Vista\B-4   8-3-18  10 in 50.liq
        Title:  PS LA Jefferson Blvd, Playa Vista,
        Subtitle:

        Surface Elev.=
        Hole No.=B4
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration= 0.43 g
        Earthquake Magnitude= 6.30

 Input Data:
        Surface Elev.=
        Hole No.=B4
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration=0.43 g
        Earthquake Magnitude=6.30
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        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: No
        * Recommended Options

        In-Situ Test Data:
        Depth    SPT      gamma    Fines
        ft                pcf      %
        ____________________________________
        1.00     14.00    120.00   NoLiq
        3.50     7.00     120.00   NoLiq
        6.00     6.00     120.00   NoLiq
        10.00    10.00    120.00   NoLiq
        15.00    5.00     120.00   NoLiq
        20.00    6.00     120.00   NoLiq
        25.00    7.00     120.00   54.00
        30.00    3.00     120.00   NoLiq
        35.00    4.00     120.00   NoLiq
        40.00    11.00    120.00   75.00
        45.00    6.00     120.00   NoLiq
        ____________________________________
```

```
Output Results:
        Settlement of Saturated Sands=2.93 in.
        Settlement of Unsaturated Sands=0.00 in.
        Total Settlement of Saturated and Unsaturated Sands=2.93 in.
        Differential Settlement=1.466 to 1.935 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        _______________________________________________________
        1.00    2.00    0.31    5.00    2.93    0.00    2.93
        2.00    2.00    0.31    5.00    2.93    0.00    2.93
        3.00    2.00    0.31    5.00    2.93    0.00    2.93
        4.00    2.00    0.30    5.00    2.93    0.00    2.93
        5.00    2.00    0.30    5.00    2.93    0.00    2.93
        6.00    2.00    0.30    5.00    2.93    0.00    2.93
        7.00    2.00    0.30    5.00    2.93    0.00    2.93
        8.00    2.00    0.32    5.00    2.93    0.00    2.93
        9.00    2.00    0.34    5.00    2.93    0.00    2.93
        10.00   2.00    0.36    5.00    2.93    0.00    2.93
        11.00   2.00    0.37    5.00    2.93    0.00    2.93
        12.00   2.00    0.38    5.00    2.93    0.00    2.93
        13.00   2.00    0.39    5.00    2.93    0.00    2.93
        14.00   2.00    0.40    5.00    2.93    0.00    2.93
        15.00   2.00    0.41    5.00    2.93    0.00    2.93
        16.00   2.00    0.42    5.00    2.93    0.00    2.93
        17.00   2.00    0.43    5.00    2.93    0.00    2.93
        18.00   2.00    0.43    5.00    2.93    0.00    2.93
        19.00   2.00    0.44    5.00    2.93    0.00    2.93
        20.00   2.00    0.44    5.00    2.93    0.00    2.93
        21.00   2.00    0.45    5.00    2.93    0.00    2.93
        22.00   2.00    0.45    5.00    2.93    0.00    2.93
        23.00   2.00    0.46    5.00    2.93    0.00    2.93
        24.00   2.00    0.46    5.00    2.93    0.00    2.93
        25.00   2.00    0.46    5.00    2.93    0.00    2.93
        26.00   0.25    0.47    0.54*   2.62    0.00    2.62
        27.00   0.25    0.47    0.54*   2.29    0.00    2.29
        28.00   0.26    0.47    0.55*   1.96    0.00    1.96
        29.00   0.26    0.47    0.54*   1.64    0.00    1.64
        30.00   0.26    0.48    0.54*   1.31    0.00    1.31
        31.00   2.00    0.47    5.00    1.30    0.00    1.30
        32.00   2.00    0.47    5.00    1.30    0.00    1.30
        33.00   2.00    0.47    5.00    1.30    0.00    1.30
        34.00   2.00    0.47    5.00    1.30    0.00    1.30
        35.00   2.00    0.47    5.00    1.30    0.00    1.30
        36.00   2.00    0.47    5.00    1.30    0.00    1.30
        37.00   2.00    0.46    5.00    1.30    0.00    1.30
        38.00   2.00    0.46    5.00    1.30    0.00    1.30
        39.00   2.00    0.46    5.00    1.30    0.00    1.30
        40.00   2.00    0.46    5.00    1.30    0.00    1.30
        41.00   0.32    0.45    0.71*   1.06    0.00    1.06
        42.00   0.32    0.45    0.71*   0.80    0.00    0.80
        43.00   0.32    0.45    0.70*   0.54    0.00    0.54
        44.00   0.31    0.45    0.70*   0.28    0.00    0.28
        45.00   0.31    0.44    0.70*   0.01    0.00    0.01
        46.00   2.00    0.44    5.00    0.00    0.00    0.00
        47.00   2.00    0.44    5.00    0.00    0.00    0.00
        48.00   2.00    0.43    5.00    0.00    0.00    0.00
        49.00   2.00    0.43    5.00    0.00    0.00    0.00
        50.00   2.00    0.43    5.00    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:  Welcome to LiquefyPro!
```

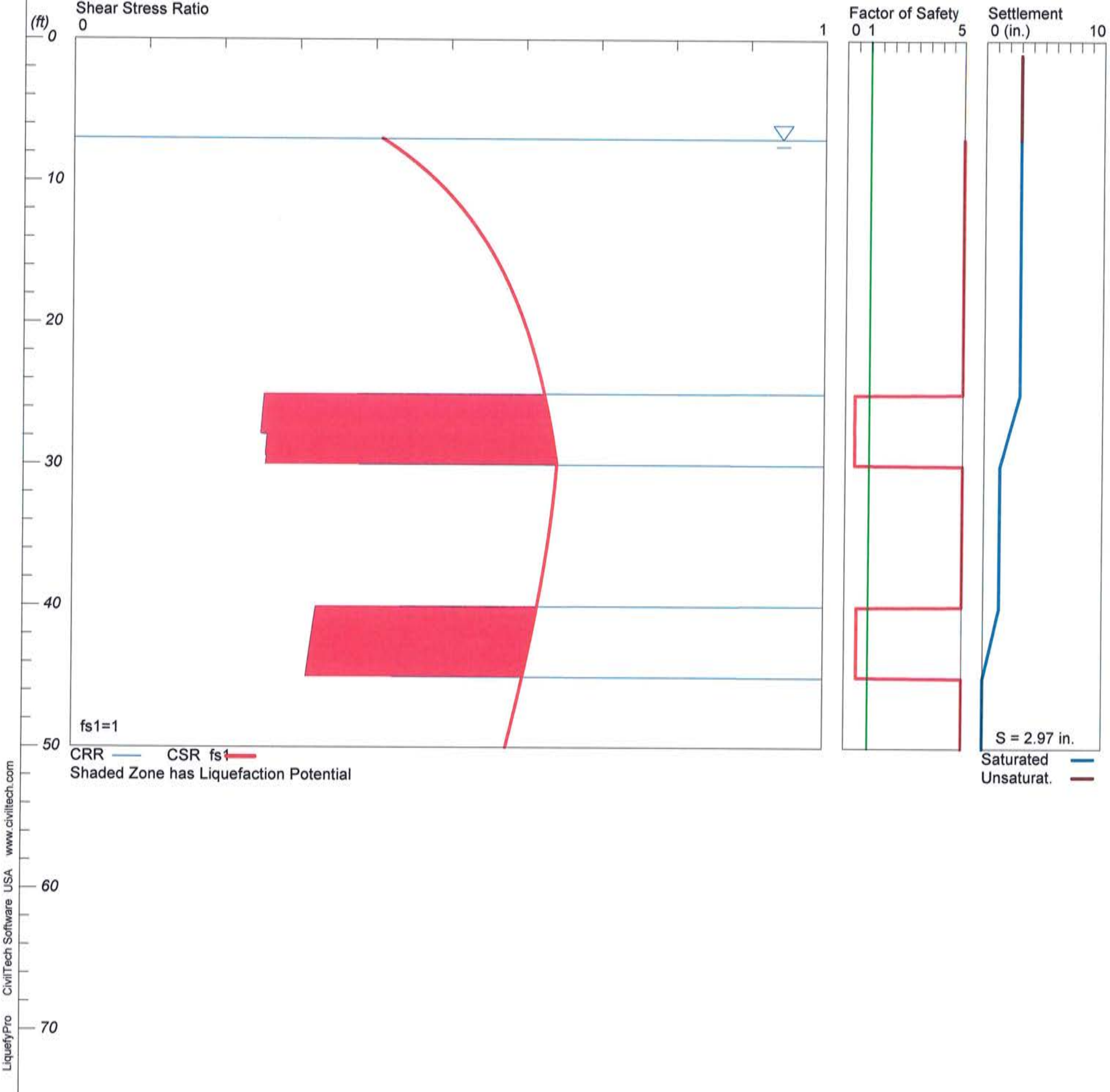
```
        1 atm (atmosphere) = 1 tsf (ton/ft2)
        CRRm            Cyclic resistance ratio from soils
        CSRsf           Cyclic stress ratio induced by a given earthquake
(with user request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        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

PS LA Jefferson Blvd, Playa Vista,

Hole No.=B4 ***Water Depth=7 ft***

Magnitude=6.3
Acceleration=0.64g



CivilTech Corporation

Plate A-1

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

                                               Copyright by CivilTech Software
                                                  www.civiltechsoftware.com


*******************************************************************************
*************************
        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   9/10/2018       4:36:31 PM

        Input File Name: P:\John Maier\Public Storage\Geo\LA 3 - Playa
Vista\B-4   8-3-18  2 in 50.liq
        Title:  PS LA Jefferson Blvd, Playa Vista,
        Subtitle:

        Surface Elev.=
        Hole No.=B4
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration= 0.64 g
        Earthquake Magnitude= 6.30

 Input Data:
        Surface Elev.=
        Hole No.=B4
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 7.00 ft
        Water Table during In-Situ Testing= 25.00 ft
        Max. Acceleration=0.64 g
        Earthquake Magnitude=6.30
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        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=1)
        10. Use Curve Smoothing: No
        * Recommended Options

        In-Situ Test Data:
        Depth    SPT      gamma    Fines
        ft                pcf      %
        ____________________________________
        1.00     14.00    120.00   NoLiq
        3.50     7.00     120.00   NoLiq
        6.00     6.00     120.00   NoLiq
        10.00    10.00    120.00   NoLiq
        15.00    5.00     120.00   NoLiq
        20.00    6.00     120.00   NoLiq
        25.00    7.00     120.00   54.00
        30.00    3.00     120.00   NoLiq
        35.00    4.00     120.00   NoLiq
        40.00    11.00    120.00   75.00
        45.00    6.00     120.00   NoLiq
        ____________________________________
```

```
Output Results:
       Settlement of Saturated Sands=2.97 in.
       Settlement of Unsaturated Sands=0.00 in.
       Total Settlement of Saturated and Unsaturated Sands=2.97 in.
       Differential Settlement=1.486 to 1.961 in.

       Depth    CRRm     CSRfs    F.S.     S_sat.   S_dry    S_all
       ft                                  in.      in.      in.
       ______________________________________________________________
       1.00     2.00     0.42     5.00     2.97     0.00     2.97
       2.00     2.00     0.41     5.00     2.97     0.00     2.97
       3.00     2.00     0.41     5.00     2.97     0.00     2.97
       4.00     2.00     0.41     5.00     2.97     0.00     2.97
       5.00     2.00     0.41     5.00     2.97     0.00     2.97
       6.00     2.00     0.41     5.00     2.97     0.00     2.97
       7.00     2.00     0.41     5.00     2.97     0.00     2.97
       8.00     2.00     0.44     5.00     2.97     0.00     2.97
       9.00     2.00     0.46     5.00     2.97     0.00     2.97
       10.00    2.00     0.48     5.00     2.97     0.00     2.97
       11.00    2.00     0.50     5.00     2.97     0.00     2.97
       12.00    2.00     0.52     5.00     2.97     0.00     2.97
       13.00    2.00     0.53     5.00     2.97     0.00     2.97
       14.00    2.00     0.54     5.00     2.97     0.00     2.97
       15.00    2.00     0.56     5.00     2.97     0.00     2.97
       16.00    2.00     0.57     5.00     2.97     0.00     2.97
       17.00    2.00     0.58     5.00     2.97     0.00     2.97
       18.00    2.00     0.58     5.00     2.97     0.00     2.97
       19.00    2.00     0.59     5.00     2.97     0.00     2.97
       20.00    2.00     0.60     5.00     2.97     0.00     2.97
       21.00    2.00     0.61     5.00     2.97     0.00     2.97
       22.00    2.00     0.61     5.00     2.97     0.00     2.97
       23.00    2.00     0.62     5.00     2.97     0.00     2.97
       24.00    2.00     0.62     5.00     2.97     0.00     2.97
       25.00    2.00     0.63     5.00     2.97     0.00     2.97
       26.00    0.25     0.63     0.40*    2.66     0.00     2.66
       27.00    0.25     0.63     0.40*    2.33     0.00     2.33
       28.00    0.26     0.64     0.40*    2.00     0.00     2.00
       29.00    0.26     0.64     0.40*    1.68     0.00     1.68
       30.00    0.26     0.64     0.40*    1.35     0.00     1.35
       31.00    2.00     0.64     5.00     1.34     0.00     1.34
       32.00    2.00     0.64     5.00     1.34     0.00     1.34
       33.00    2.00     0.64     5.00     1.34     0.00     1.34
       34.00    2.00     0.64     5.00     1.34     0.00     1.34
       35.00    2.00     0.63     5.00     1.34     0.00     1.34
       36.00    2.00     0.63     5.00     1.34     0.00     1.34
       37.00    2.00     0.63     5.00     1.34     0.00     1.34
       38.00    2.00     0.62     5.00     1.34     0.00     1.34
       39.00    2.00     0.62     5.00     1.34     0.00     1.34
       40.00    2.00     0.62     5.00     1.34     0.00     1.34
       41.00    0.32     0.61     0.52*    1.09     0.00     1.09
       42.00    0.32     0.61     0.52*    0.82     0.00     0.82
       43.00    0.32     0.61     0.52*    0.55     0.00     0.55
       44.00    0.31     0.60     0.52*    0.28     0.00     0.28
       45.00    0.31     0.60     0.52*    0.01     0.00     0.01
       46.00    2.00     0.59     5.00     0.00     0.00     0.00
       47.00    2.00     0.59     5.00     0.00     0.00     0.00
       48.00    2.00     0.59     5.00     0.00     0.00     0.00
       49.00    2.00     0.58     5.00     0.00     0.00     0.00
       50.00    2.00     0.58     5.00     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:  Welcome to LiquefyPro!
```

```
        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.

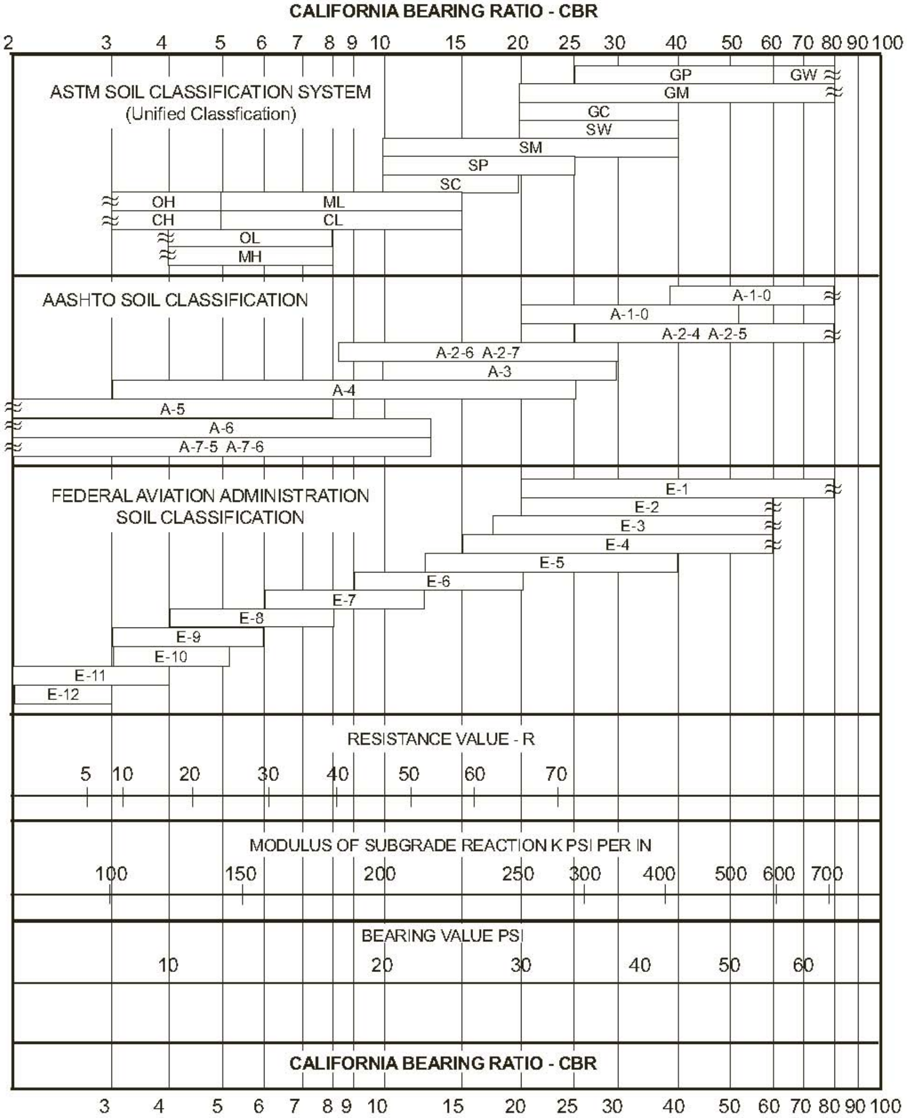
Laboratory Testing

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.

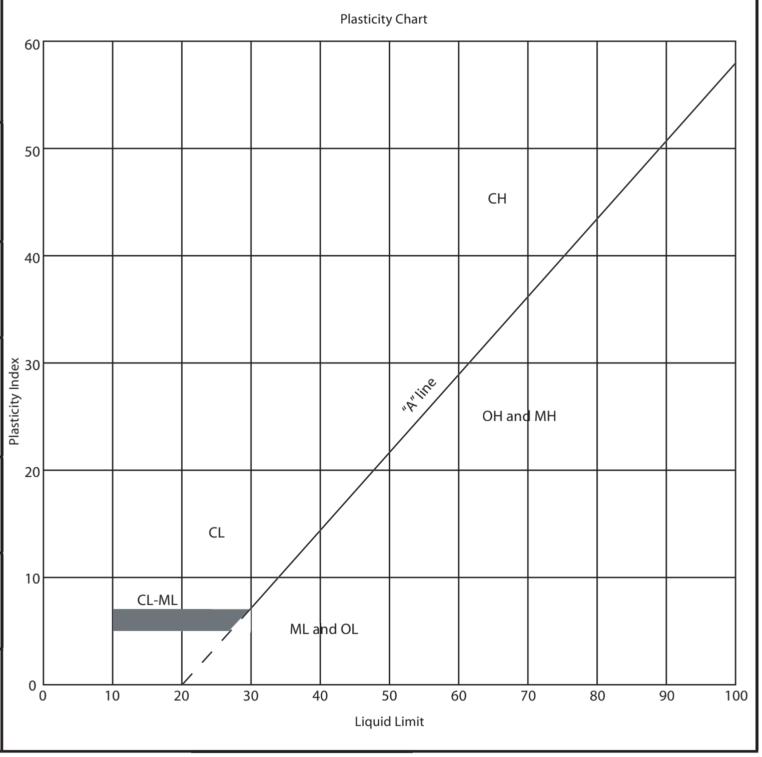
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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			

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[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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