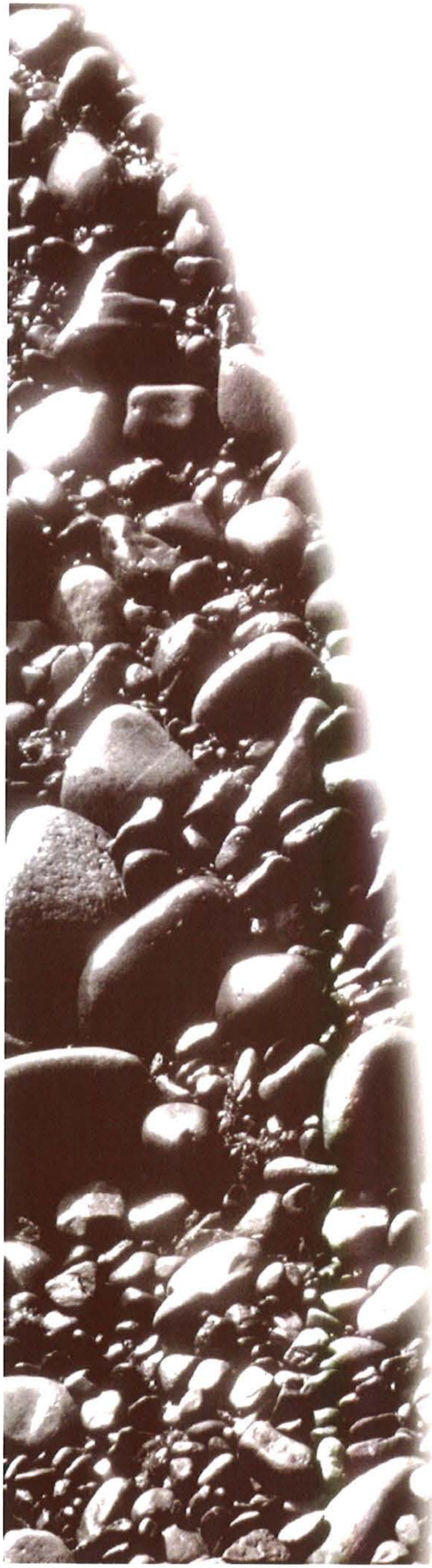

YELLOW

Geotechnical Engineering Exploration and Analysis

Proposed Public Storage Redevelopment
Two New Three Story Buildings
NWC W. Capitol Expressway and Snell Avenue
San Jose, California

Prepared for:

Public Storage
Glendale, California

December 20, 2017
Project No. 2G-170813

GILES
ENGINEERING ASSOCIATES, INC.

December 20, 2017

Public Storage, Inc.
701 Western Avenue
Glendale California 91201

Attention: Mr. Mark Kennedy
Development Manager

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Public Storage Redevelopment
Two New Three Story Buildings
NWC W. Capitol Expressway and Snell Avenue
San Jose, California
Project No. 2G-1708013

Dear Mr. Kennedy:

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

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

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

REGISTERED PROFESSIONAL ENGINEER
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SAN JOSE, CALIFORNIA
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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED PUBLIC STORAGE REDEVELOPMENT
TWO NEW THREE STORY BUILDINGS
NWC W. CAPITOL EXPRESSWAY AND SNELL AVENUE
SAN JOSE, CALIFORNIA
PROJECT NO. 2G-1708013

EXECUTIVE SUMMARY OUTLINE

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

Subsurface Conditions

- Site Class designation "D" is recommended for seismic design considerations.
- Our review of the *Geologic Map of San Jose East Quadrangle, California* prepared by California Geological Survey indicates that the subject site is located within a designated Liquefaction Hazard Zone and in an area where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent liquefaction induced displacements, such that mitigation is required. However, based on our liquefaction analysis seismic induced settlement within the site is within tolerable limits for the planned development.
- Our review of the *Geologic Map of San Jose East Quadrangle, California* prepared by California Geological Survey indicates that the subject site is underlain by Alluvial fan deposits with fine grained facies.
- Existing pavement encountered within our test borings consisted of approximately 2 to 5 inches thick asphaltic concrete over 2 to 8 inches of aggregate base. Based on our visual observation, the existing pavement is in fair condition.
- Fill and possible soils were encountered within our test borings to depths of about 1.5 to 3.5 feet below existing grades. These soils generally consisted of moist, stiff sandy clay, silty clay and firm silty sand with some gravel.
- Native soils encountered underneath the fill and possible fill, generally consisted of moist to wet, soft to medium stiff silty clay and sandy clay to the maximum depth explored (51.5 feet). A very dense sand with gravel layer was encountered within test boring B-6 at depths of about 45 to 50 feet below existing grade.
- Groundwater was encountered at depths of about 20.67 to 21.5 feet below existing ground surface during our subsurface investigation. The site may however be subject to a shallow perched water table during wet periods.

Site Development

- The proposed development will consist of constructing two, 3-story buildings with about 134,712 square feet and 132,912 square feet with parking stalls and new drive lanes.
- Due to the presence of existing fill and variable strength characteristics of the near surface on-site soils, and the likely disturbance of the subgrade soils during removal of existing building foundations and floor slab, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (3 feet minimum where possible) be over-excavated to a depth of at least 2½ feet below existing grade or planned pad grade, and at least 1.5 feet below the bottom of foundations and floor slab whichever is lower in elevation, some overexcavation may be required and should be budgeted due to the existing fill and demolition disturbance.

Compacted crushed aggregate material (2-to-4 inch diameter) about 6 inches thick should be placed at the bottom of the excavation followed by a layer of geogrid, such as Tensar Biaxial Geogrid TX140 or better, and then by imported well graded granular material compacted in place to at least 90% of the soils maximum dry density as determined by Modified Proctor (ASTM D-1557).

Building Foundation

- The proposed structures may be supported by a shallow spread footing foundation system and/or a mat/slab supported on a minimum thickness structural compacted fill layer. The maximum, net allowable soil bearing pressure may range from 3,000 to 4,000 pounds per square foot (psf) depending on the structural fill thickness. A maximum modulus of subgrade reaction (k_s) of 65 pounds per square inch per inch (psi/in.) may be used with a mat/slab foundation system.
- Steel reinforcing should be per the structural engineer.

Building Floor Slab

- The floor may be part of the foundation system where it is designed as a mat/slab, or as a conventional slab-on-grade independent of the foundations.
- Where the floor is designed as a conventional slab-on-grade, independent of the foundations it may be designed based on a maximum modulus of subgrade reaction (k_s) of 125 psi/in.
- The slab for the buildings 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 to be directly below the floor slab or base course throughout the entire floor or mat/slab area.

Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 6 and 8 inches of base course in parking stall and drive lane areas, respectively.
- Portland Cement Concrete: 7 inches in thickness in high stress areas such as entrance/exit aprons lane and in trash enclosure loading zone with a 4 inch granular base.

YELLOW – This site has been given a Yellow designation, due to increased costs associated with building pad preparation in consideration of overexcavation due to razing existing structures and existing fill in preparation of a structural fill layer below the structure and the disturbance and water sensitivity of the subgrade soil, and the recommended stabilization of the subgrade.

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report. The scope of work performed for this report was consistent with the scope of work outlined within Proposal No. 2GP-1708017.

The scope of services authorized for this project included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and a geotechnical engineering analysis to provide criteria for preparing the design of the foundation and floor slab for the proposed development. Geotechnical-related recommendations are also provided for the proposed parking lot improvement. Site preparation recommendations are also given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include the weather before and during construction, the water table at the time of construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development.

2.0 SITES AND PROJECT DESCRIPTION

2.1 Site Description

The subject site is occupied by two adjacent Public Storage facilities (231 W. Capitol Expressway and 3911 Snell Avenue) and located near the northwest corner of Capitol Expressway and Snell Avenue in the city of San Jose, California. For purposes of this report, Project North has been defined parallel to Snell Avenue as shown on Figure 1. The site is bounded on the south by W. Capitol Expressway, on the east by Snell Avenue, on the north by a mobile home community, on the west by a multi-unit residential development, with a Shell Gas Station located to the southeast of the site. The site is located at 37.2769 Latitude, -121.8433 Longitude.

The site is currently occupied by several single story storage structures (A to P) and parking and drive lanes. The existing buildings and pavement are in good condition. Topography within the site is relatively level. Our review of the survey map performed by Lars Anderson and Associates, Inc. for the subject site indicated existing elevations within the proposed new building areas range from about elevation 158 feet to 161 feet.

2.2 Proposed Project Description

Based on the preliminary site plans provided to us, existing buildings A, D E, F, K, L, M, N, O and P will be demolished and removed to accommodate the construction of new two separate three-story storage buildings, parking stalls and drive lanes.

Two free-standing, three-story structure will be constructed in the area shown on the *Test Boring Location Plan*. Details of the proposed building were not provided, but it is understood that the building will be a steel-frame structure with a masonry, concrete, or metal exterior, and a steel bar-joist and metal-deck roof. Interior and perimeter columns will support the structure. Based on other Public Storage projects, interior columns are expected to be on a 10-foot grid. Furthermore, it is understood that a mat foundation (which will serve as the first floor) is planned for the building. It is also understood that the planned foundation includes a continuous footing (or grade beam) at the perimeter of the building, with isolated-column footings at various locations within the building for cross-bracing and for other structural reasons. The maximum foundation load from columns is estimated to be about 40 to 50 kips, and the maximum first-floor load is assumed to be 125 pounds per square foot (psf). The proposed building will not have a basement, but will have elevator pits. Elevator pits are expected to be cast-in-place concrete structures that are a maximum of 4 feet deep.

The proposed improvements will assumedly include new parking areas and drives. New pavement is expected to consist of asphalt-concrete, except in high-stress areas, where Portland cement concrete (PCC) pavement is expected.

Preliminary project information did not indicate the planned finished floor elevation for the proposed new structure. However, it is our understanding that the finish floor elevation of the new buildings will closely match the elevation of existing building floor elevations of about El. 159. Therefore, site grading is anticipated to consist of minor grading, in order to establish the necessary anticipated finish grade elevations, exclusive of site preparation and over-excavation requirements necessary to create a stable site suited for the proposed development.

The traffic loading for the driveway and parking lot areas is understood to predominantly consist of automobiles and recreational vehicles, with occasional heavy trucks resulting from deliveries and trash collection. Pavement designs are based on a 20-year design period. The parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.0 for the automobile traffic parking stalls (light duty) and a TI of 5.5 for automobile drive lane areas (medium duty).

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration was performed by representatives of our firm and consisted of the drilling of ten (10) test borings (B-1 to B-10) to depths of approximately 5 to 51.5 feet below existing ground surface utilizing a hollow stem auger drill rig. A previous subsurface exploration was performed by this firm at the subject site on September 2013 and involved drilling of four (4) test borings to depths of about 16.5 feet to 46.5 feet. The approximate test boring locations (current and previous) are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures 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 included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the boring. 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 exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

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 plastic containers and transported to our laboratory for testing.

3.2 <u>Subsurface Conditions</u>

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.

<u>Pavement</u>

Existing pavement encountered within our test borings consisted of approximately 2 to 5 inches thick asphaltic concrete over 2 to 8 inches of aggregate base. Based on our visual observation, the existing pavement is in fair condition.

<u>Soil</u>

Our review of the *Geologic Map of San Jose East Quadrangle, California* prepared by California Geological Survey (2000) indicated that the subject site is underlain by fine grained alluvial fan deposits from the Holcene era.

Fill and possible soils were encountered within our test borings to depths of about 3.5 to 5 feet below existing grades. These soils generally consisted of moist, stiff sandy clay, silty clay and firm silty sand with some gravel.

Native soils encountered underneath the fill and possible fill, and underneath the pavement generally consisted of moist to wet, soft to medium stiff silty clay and sandy clay to the maximum depth explored (51.5 feet). Very dense sand with gravel layer was encountered within test boring B-6 at depths of about 45 to 50 feet below existing grade.

Groundwater

Groundwater was encountered at depths of about 20.67 to 21.5 feet below existing ground surface during our subsurface investigation. The historic high groundwater elevation is approximately 20 feet below the ground surfaced per data for the San Jose East Quadrangle published by California Geological Survey (CGS). However, 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 at which point the water tables or perched water table may rise to within several feet of the ground surface. Irrigation of landscape areas on or adjacent to the site can 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 within the subject site. Three percolation tests were conducted at the subject site (designated as P-1 to P-3) and involved the drilling of one test boring utilizing a hollow-stem auger drill rig with an outside diameter of approximately 6 inches. A two-inch perforated pvc pipe was installed inside the boring and pea gravel was used as filter pack around the outside diameter of the pipe. A percolation test was performed at a depth of approximately 5.0 feet below the existing ground surface. Testing involved presoaking the test hole and filling the test hole with water, and recording the drop in the water surface. 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 results obtained from our percolation testing are summarized below.

TABLE 1 – INFILTRATION TEST RESULTS

Test Hole	Test Depth[1] (feet)	Infiltration Rate (in/hr)	Soil Type
P-1 (B-9)	5.0	0.15	Silty Clay
P-2 (B-10)	5.0	0.06	Silty Clay
P-3 (B-3)	5.0	0.02	Silty Clay

1) Depth is referenced to the existing surface grade at the test location.
2) in/hr is inch per hour

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

4.0 LABORATORY TESTING

Several laboratory tests were performed on selected samples considered representative of those encountered in order to evaluate the engineering properties of 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 2216-05. The results of these tests are included in the Test Boring Logs enclosed in Appendix A.

Sieve Analysis

Sieve Analyses (Passing No. 200 Sieve) were performed on selected samples from various depths within test borings B-1 and B-6 to assist in soil classification and aid in the liquefaction analysis. These tests were performed in accordance with Test Method ASTM D 1140-00 (Reapproved 2006). The results of these tests are presented in Test Boring Logs, Appendix A.

Atterberg Limits

The Atterberg limits (liquid limit, plastic limit and plasticity index) were determined for representative samples of the clayey on-site soils in accordance with Test Method ASTM D 4318-05. The results of the Atterberg Limits indicated Plastic Index that ranged from 14to 29 (medium to high plasticity) and are included on the Test Boring Logs enclosed in Appendix A.

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered within the proposed buildings, a composite sample collected from Test Borings B-1 and B-2 (1 to 5 feet) was subjected to Expansive Index (EI) testing per ASTM D 4829-03. The result of our expansion index (EI) testing indicates that the near surface sample in the area of the proposed addition has a medium expansion potential (EI=62).

Consolidation Test

Potential swell, collapse and settlement estimates under anticipated load were made on the basis of one-dimensional consolidation test. These tests were performed in general accordance with Test Method ASTM D 4546. The test samples were inundated near the on-site overburden pressure in order to evaluate the sudden increase in moisture condition (swell or collapse potential). Results of these tests indicated that on-site soils are slightly compressible and are graphically presented as Figure 2 and 3 and included 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 and B-2 1 to 5 feet**
pH	8.70
Chloride	50 ppm
Sulfate	0.0480%
Resistivity	2,100 ohm-cm

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

The results of limited in-house testing of soil pH and resistivity were determined in accordance with California test Method No. 643 and indicated that the site soils are strongly alkali with respect to pH and soil resistivity was found to possess a moderate degree of corrosivity. 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. We recommend that the results be evaluated by a corrosion engineer to determine if special corrosion protection is needed for this site.

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample with California Test Method No. 417. Our laboratory test data indicated that near surface soils contain approximately 0.0480 percent of water soluble sulfate. 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 on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 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.

5.2 Seismic Hazard Zones

Our review of the published Seismic Hazard Evaluation Report for the San Jose East Quadrangle indicates that the subject site is located within a designated Liquefaction Hazard Zone and in an area where historical occurrence of liquefaction, or local geological, geotechnical, and ground water conditions indicate a potential for permanent liquefaction induced displacements, such that mitigation might be required.

General types of ground failures that might occur as a consequence of severe ground shaking typically include landsliding, 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, laboratory testing and the seismic designation for this site, all of the above effects of seismic activity are considered unlikely at the site.

5.3 Landslide Hazards

The subject site does not lie within the designated Landslide Hazard Zone based on our review of the published Seismic Hazard Evaluation Report for the San Jose East Quadrangle. Since the subject site is generally level and not located near unstable slope, mitigation of landslide hazards is not necessary for the site.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

From a soils engineering point of view, the subject property is considered geotechnically suitable for the proposed new improvements provided the following recommendations are incorporated in the design and construction of the project. We recommend that Giles Engineering Associates, Inc. be involved in the review of the grading and foundation plans for the site to ensure our recommendations are interpreted correctly. Based on the results of our review, modifications to our recommendations or the plans may be warranted.

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is therefore considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2016 California Building Code (CBC) and applicable local codes. Based upon the encountered subsurface soils, a Site Class D is recommended for design.

According to the maps of known active fault near-source zones at the subject site (37.2769 Latitude; 121.843 Longitude), Monte Vista-Shannon, Calaveras and San Andreas faults are the closest active faults and are located approximately 4.43, 8.35 and 11.15 miles from the site, respectively, with anticipated maximum moment magnitude (Mw) of 6.5, 7.03 and 8.05..

Within the International Code Council's 2015 International Building Code (IBC), 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 program Java Ground Motion Parameter Calculator- Version 5.10.0 written by the ICC.

CBC 2016, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.5(3) for 0.2 second)	1.500
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.5(4) for 1.0 second)	0.600
Site Coefficient, F_a (Table 1613.5.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.5.3 (2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.500
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.900
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	1.000
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.600

Liquefaction

According to the Seismic Hazard Zones map for the San Jose East Quadrangle published by the California Geological Survey (CGS), the site is located within an area that has been designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. Therefore, a site liquefaction evaluation consistent with the guidelines contained in CDMG Special Publication 117A (2008) along with a report by Southern California Earthquake Center (SCEC) has been performed as part of the current investigation.

The liquefaction analysis was performed utilizing the computer software program LiquefyPro and based on the 2016 CBC. For this analysis we used the soil profile identified with boring B-1 and B-6. The site acceleration (PGA_M) of 0.508g was obtained from the USGS website and determined from ASCE-07 based on a 2% probability of exceedance in 50 years, or an actual return period of 2,475 years. The predominant earthquake magnitude (Mw) at the site is 6.69 based upon a deaggregation analysis for a return period of 2,475 years, obtained from the USGS website. Input parameters for blow count data were corrected for borehole diameter, sampling type, automatic hammer type, and depth.

Groundwater was encountered in the borings at depths ranging from 20.67 to 21.5 feet below existing grade. Based on a review of the CGS Seismic Hazard Zone Report, Hayward Quadrangle, the historic high groundwater elevation for this site is approximately 20 feet below grade. Therefore, a groundwater depth of 20 feet was used for our analysis.

Soils 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. As noted within SP 117A, 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.

Test Boring No. & Depth	Liquid Limit (LL)	Plastic Index (PI)	In-situ Moisture<85% of LL	Wc/LL
B-1 @ 20'	47	29	29 <40	0.62
B-1 @ 30'	48	26	32 < 41	0.67

Test Boring No. & Depth	Liquid Limit (LL)	Plastic Index (PI)	In-situ Moisture<85% of LL	Wc/LL
B-6 @ 20'	36	14	28 < 30.6	0.78
B-6 @30'	53	25	31<45.1	0.58
B-6 @ 50'	40	16	15<34	0.38

Based on Bray and Sancio (2006) criteria, the fine grained soils when plotted for Plastic Index and in-place moisture content / Liquid Limit indicated not susceptible for liquefaction potential.

Our liquefaction study was based on the NCEER procedure (Youd & Idriss, 1998) using a peak ground acceleration of 0.508 g and an earthquake magnitude of 6.69. 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 were accounted in the program for hammer energy ratio, borehole diameter and sampling method. A conservative historic high groundwater of 10 feet was used in our liquefaction analysis. The liquefiable layers at the location of borings B-1 and B-6 are presented graphically in Plates A1 and A2 of Appendix A. The computer output files are also included.

In order to estimate the amount of post-earthquake settlement and/or liquefaction, methods proposed by Tokimatsu and Seed (1987) were used for the calculations. Based on our analysis and under the current site conditions with an assumed high water table of 20 feet, we estimate that the maximum total seismic-induced ground settlement at the site would be about 0.34 and 1.31 inches during the design level earthquake. Borings B-1 and B-6 are about 325 feet apart and have a differential settlement of 0.97 in. Assuming a factor of safety of 3.0, the maximum differential settlement is estimated to be about ⅓ inch over a horizontal span of 30 feet.

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

Liquefaction–Induced Potential for Surface Manifestation

Based on our review of the relationships between the thickness of potentially liquefiable soil layers relative to the thickness of non-liquefiable soil layers developed by Ishihara (1985) and soil classification, it is our opinion that surface manifestations resulting from soil liquefaction at this site is not likely and should not be considered a design constraint for the project.

6.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations and the results of our laboratory testing and liquefaction analysis. Moist to wet soil conditions, as well as low to medium plasticity clay soils, were encountered within the near surface investigation. It is expected that similar conditions are likely to be encountered during grading operations. Grading operations may require provisions for drying of soils prior to compaction. In addition, due to the presence of moist to very moist, clayey on-site soils at the proposed remedial grading depths, the loads imposed by heavy rubber-tired equipment during grading may induce localized pumping of the subgrade that would require stabilization prior to fill placement. Grading contractor should therefore include contingencies for air-drying of excessively moist soil, as well as the stabilization of excavation bottoms in their bids. Imported soils may be required if onsite soils cannot be air-dried on site due to space, time constraints, or weather. A mud-slab or stabilization layer of medium to coarse crushed aggregate is therefore be required at the subgrade as discussed under *Building Area* heading.

Site Clearing

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

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

Existing Utilities

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

Building Area

The consolidation test results as well as the relatively low blow counts indicated the presence of compressible and low to moderate strength on-site clayey soils that are susceptible to a moderate degree of consolidation under the weight of the new building and new compacted fills where existing grades are increased.

Due to the presence of variable strength characteristics of the near surface on-site soils, and the likely disturbance of the subgrade soils during removal of existing building foundations and floor slab, it is recommended that the soils within the proposed new building areas and an appropriate distance beyond (3 feet minimum where possible) be over-excavated to a depth of at least 2½ feet below existing grade or planned pad grade, and at least 18 inches (1.5 feet) below the bottom of foundations and floor slab whichever is lower in elevation. Compacted crushed aggregate material (2-to-4 inch diameter) about 6 inches thick should be placed at the bottom of the excavation followed by a layer of geogrid, such as Tensar Biaxial Geogrid TX140 or better, and then by imported well graded granular material compacted in place to at least 90% of the soils maximum dry density as determined by Modified Proctor (ASTM D-1557). 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. Some overexcavation may be required due to disturbance from existing building demolition and from unsuitable materials, such as existing fill at all boring and especially at borings with deeper existing fill.

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

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

Proofroll and Compact Subgrade

Following site clearing, the subgrades within the proposed pavement areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded dump-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 less than 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 material may be reused as structural compacted fill (if needed) within the proposed building and pavement area 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. 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.

Import Structural Fill

The soils imported to the site for use as structural fill should consist of very low to low expansive soils (EI less than 51). Material 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.

Subgrade Protection

The near surface soils that are expected to comprise the subgrade are sensitive to water. Unstable soil conditions will develop if these 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 expected.

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 pad 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 in accordance with the enclosed "Guide Structural Fill Specifications". A representative of the project geotechnical engineer should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

6.3 Construction Considerations

Construction Dewatering

Groundwater was encountered at depths of about 20.67 to 21.5 feet below existing ground surface during our subsurface investigation. Therefore, groundwater is not expected to impact shallow excavations for footings and utilities. However, the site may be susceptible to the development of shallow perched water conditions during wet periods. 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 may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Shallow excavations may be adequately sloped for bank stability while deeper excavations or excavations in areas where adequate back sloping is not possible, some form of sheeting or shoring may be required.

6.4 Foundation Recommendations

Upon completion of the recommended building pad preparation and completion of site grading, it is our opinion the proposed structures may be supported by a shallow foundation system underlain by a minimum 18 inches (1.5 feet) thick structural fill layer. Strip footings may be used to support the bearing walls and isolated column spread footings designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf). Minimum footing widths are recommended to be 18 and 24 inches for walls and columns, respectively, regardless of the calculated soil bearing pressure. For high column loads, such as areas with 20 x 20 foot bays, the soil bearing capacity for spread footings may be increased by increasing the structural fill layer thickness. A 4,000 psf maximum allowable bearing capacity is recommended where the structural fill layer below the footing is at least one-half the column footing width in thickness. A mat/slab may also be used to support the structure designed for a 3,000 psf maximum allowable soil bearing pressure. The maximum modulus of subgrade reaction (k_s) for the mat/slab and strip footings designed as a mat and beam on an elastic foundation is 65 pounds per square inch per inch (psi/in.). The recommended allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads. The structural fill layer characteristics, including its lateral extension beyond the footing edge, should be in accordance with the requirements of the enclosed Structural Fill Specifications

Reinforcing

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

Lateral Load Resistance

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

A coefficient of friction of 0.27 may be used with dead load forces for footings placed on newly placed compacted fill soil. An allowable passive earth pressure of 225 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against newly placed structural fill. The maximum recommended allowable passive pressure is 1,500 psf.

Bearing Material Criteria

Evaluation of the structural fill subgrade and therefore the indirect foundation bearing soils is recommended to be performed by the geotechnical engineer at the time of construction of the structural fill layer. Evaluation is recommended to be performed to a sufficient depth below the structural fill subgrade to verify the bearing suitability of the foundational soils. If unsuitable bearing soil is encountered within the foundation influence zone, it should be removed and replaced with a properly compacted engineered fill.

Soil suitable to serve as the foundation bearing grade should exhibit at least a loose relative density (average N value of at least 9) for non-cohesive soils or possess a stiff consistency (average unconfined compressive strength of 1.5 tsf) for cohesive soils for the recommended allowable soil bearing pressure. For design and construction estimating purposes, suitable bearing soils are expected to be encountered at nominal foundation depths following site grading. 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.

Foundation Embedment

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

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than 1 and ½ inch, respectively, for static conditions. The estimated differential movement is anticipated to result in an angular distortion of less than 0.002 inches per inch on the basis of the ½ inch differential movement occurring over a minimum span of 20 feet. The maximum estimated total and differential movement is considered within tolerable limits for the proposed structures provided it is considered in the structural design.

6.5 Floor Slab Recommendations

Subgrade

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

Design

The floor may be designed as a conventional slab-on-grade where the floor is independent of the foundations, such as areas with a 20 x 20 foot bay spacing; or as a mat/slab where the floor also serves as the foundation system. Design parameters for the mat/slab are provided in section 6.4.

The ground floor of the proposed structure may be designed and constructed as a conventional slab-on-grade where the floor slab is independent of the foundation system. The at-grade floor may be designed as a "Mat on Elastic Foundation" using a Modulus of Subgrade Reaction (k_s) of 125 pounds per cubic inch (pci) where the slab provides no structural support for the interior load bearing walls and/or columns. The design of the slab is recommended to be performed by the project structural engineer to ensure proper reinforcing and thickness.

The floor slab whether designed as a mat/slab or conventional slab-on-grade is recommended to be underlain by a 4-inch thick layer of compacted granular material. A minimum 15-mil vapor retarder is recommended to be directly below the floor slab or base course throughout the entire floor area. 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 the base course has sharp, angular aggregate, capable of puncturing the vapor barrier, it should be protected with a geotextile (or by other means).

Estimated Settlement

With proper site preparation and construction monitoring, the total and differential settlements of a slab-on-grade, are estimated to be less than ¾ and ¼ inches across a 20 foot span, respectively. Therefore, settlements are on the order of the estimates for the building perimeter foundation.

6.6 Pavement Recommendations

Asphalt Pavement

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

New Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of existing on-site soils that exhibit a low to medium expansion potential. The anticipated subgrade soils are classified as a fair subgrade material with an estimated R-value of 5 to 10 when properly prepared based on the Unified Soil Classification System designation of CL. An R-value of 5 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of San Jose 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	Drive Lanes	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	6	8	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 7 inches thick containing No. 4 bars at 18-inch on-center both ways placed at mid-height. The pavement should be constructed in accordance with Section 40 of the CALTRANS Standard Specifications. A minimum 4-inch thick layer of base course (CALTRANS Class 2) is recommended below the concrete pavement. This base course should be compacted to at least 95% of the material's maximum dry density.

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

General Considerations

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

6.7 Recommended Construction Materials Testing Services

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

6.8 Basis of Report

This report is based on Giles' proposal, which is dated August 22, 2017 and is referenced by Giles' proposal number 2GP-1708017. 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.

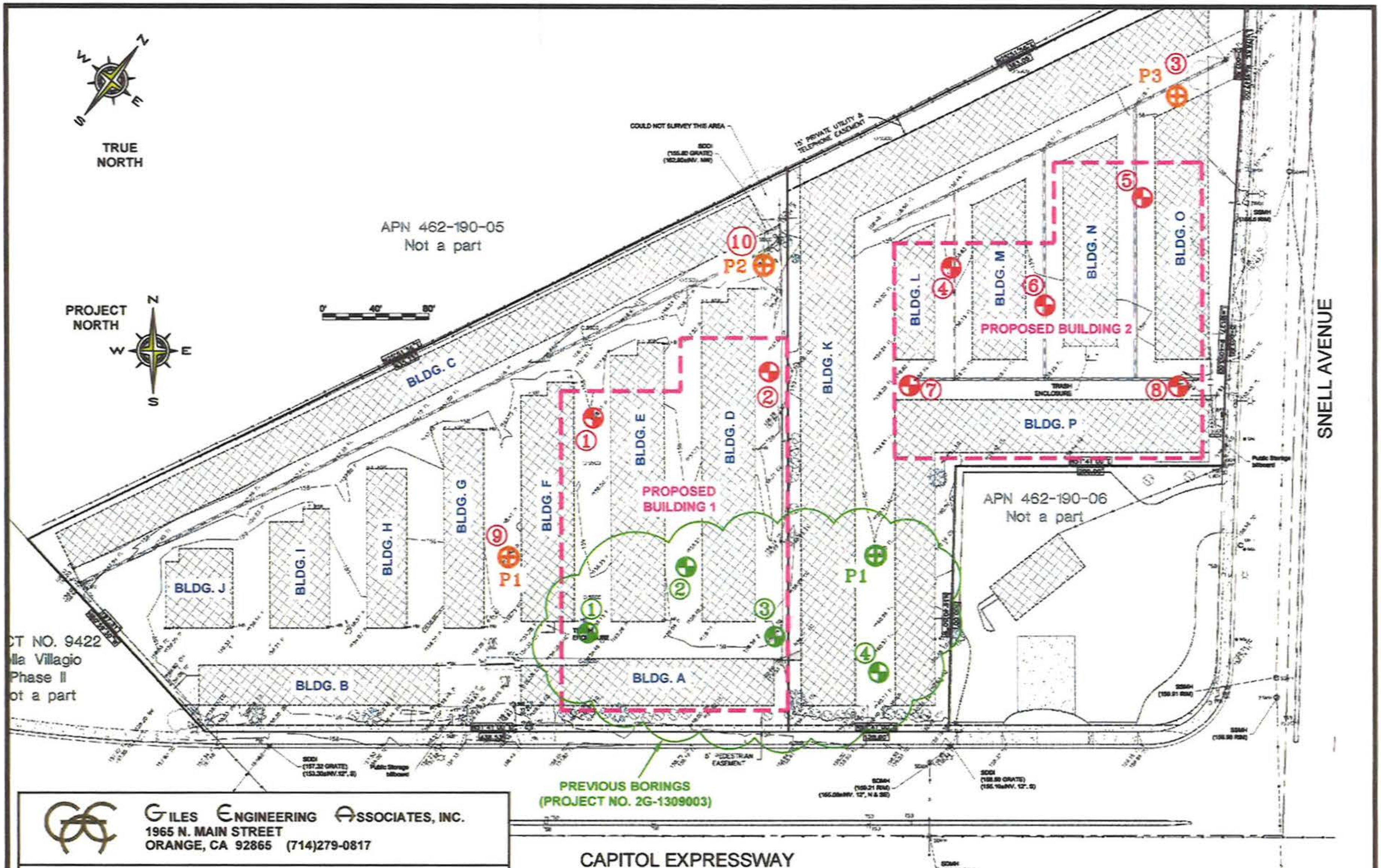
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *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.



GILES ENGINEERING ASSOCIATES, INC.
1965 N. MAIN STREET
ORANGE, CA 92865 (714)279-0817

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED PUBLIC STORAGE REDEVELOPMENT
NWC, W. CAPITAL EXPRESSWAY AND SNELL AVENUE
SAN JOSE, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
JLM	JSZ	approx. 1"=100'	11-13-13	--
PROJECT NO.: 2G-1708013			CAD No. 2g1708013-blp	

LEGEND:

1 GEOTECHNICAL TEST BORING

P1 PERCOLATION TEST BORING

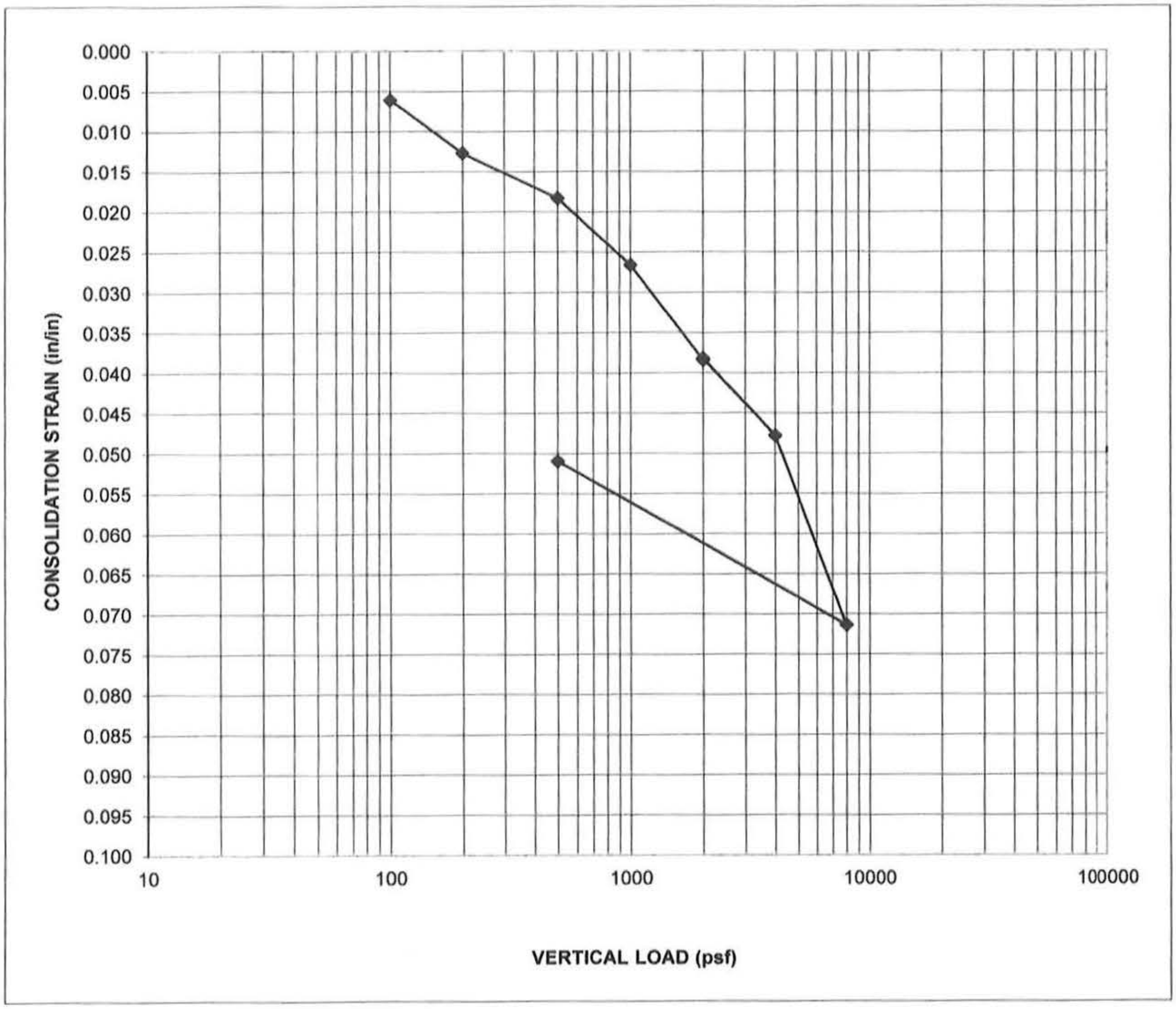
NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "TOPOGRAPHIC SURVEY", PREPARED BY LARS ANDERSEN & ASSOCIATES, INC.

3.) PROPOSED BUILDINGS ARE APPROXIMATE BASED ON THE "SITE PLAN - SCHEME A (PHASE 2)", DATED 6-29-17, PREPARED BY KSP STUDIO.

COLLAPSE TEST ASTM D5333



Classification	Silty Clay			
Boring No.		B-2		
Sample No.		3-CS	Initial Moisture Content (%)	22.7
Depth (ft.)		6.0 - 7.5	Final Moisture Content (%)	22.2
Elevation			Natural Density (pcf)	118.8
Liquid Limit			Initial Dry Density (pcf)	96.8
Plastic Limit			Final Dry Density (pcf)	102.0
Specimen Diameter (in.)		2.42		
Initial Specimen Thickness (in.)		1.00		

Sample inundated at 2000 psf pressure

Project:	PS San Jose
Client:	Public Storage, Inc.
Project No.:	2G-1708013
Figure No.:	2

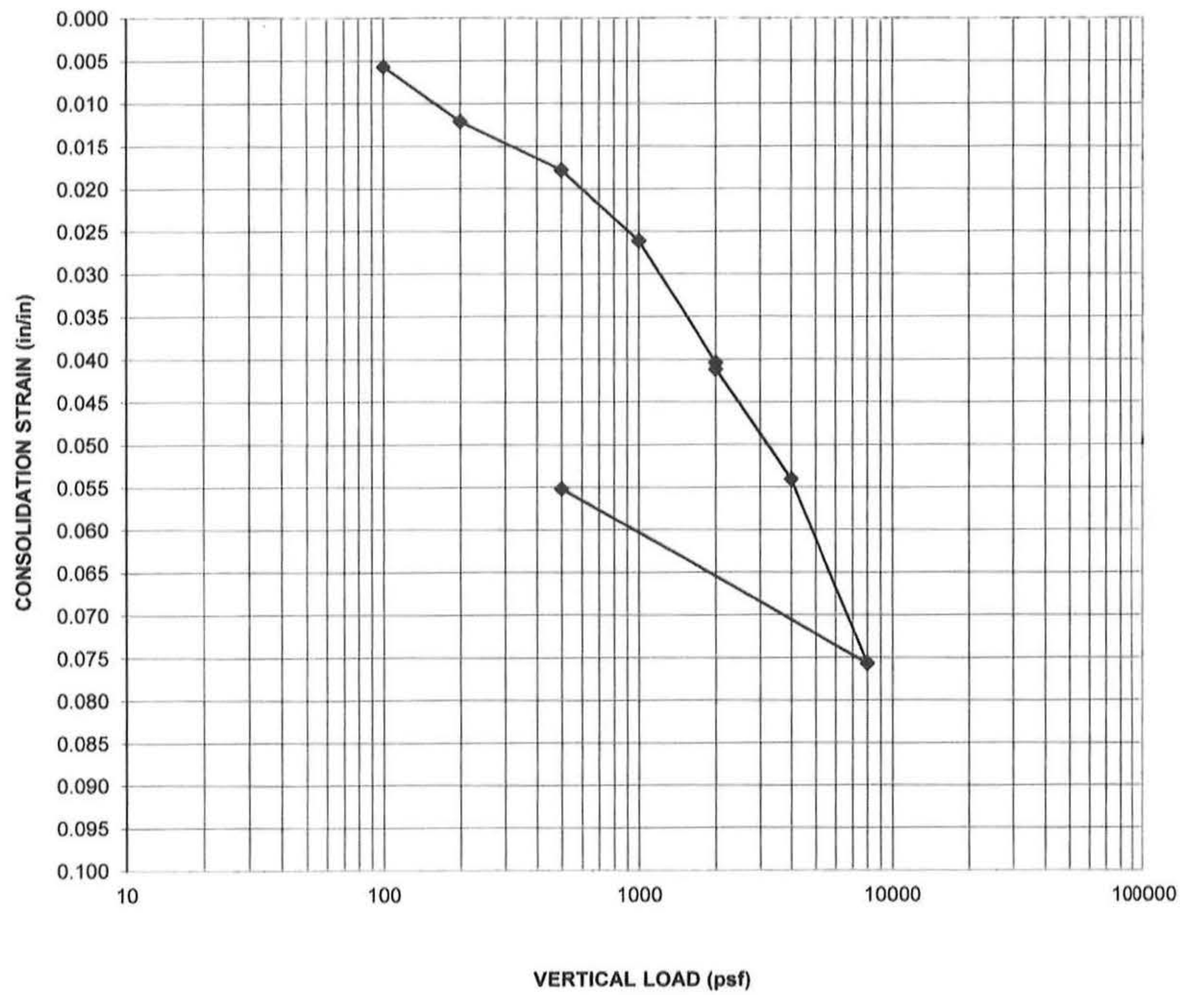
GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-

1965 NORTH MAIN STREET, ORANGE, CALIFORNIA

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

COLLAPSE TEST ASTM D5333



Classification	Silty Clay		
Boring No.	B-5		
Sample No.	3-CS	Initial Moisture Content (%)	19.9
Depth (ft.)	6.0 - 7.5	Final Moisture Content (%)	20.1
Elevation		Natural Density (pcf)	121.8
Liquid Limit		Initial Dry Density (pcf)	101.6
Plastic Limit		Final Dry Density (pcf)	107.6
Specimen Diameter (in.)	2.42		
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 2000 psf pressure

Project:	PS San Jose
Client:	Public Storage, Inc.
Project No.:	2G-1708013
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

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
B- 1		
SURFACE ELEVATION: 157.9 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/10/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 3 inches of aggregate base			1-SS	9		3.25		20		EI=62 (medium)
Dark Brown fineSandy Clay, some pieces of brick - Moist to Very Moist (Fill)			2-SS	14		4.5+		18		P_{200}=90%
Light Brown to Olive Brown fine Sandy Clay - Very Moist (Native)		150	3-SS	9		4.5		21		
	10		4-SS	8				21		
		140	5-SS	6				28		P_{200}=88%
Light Brown Sandy Clay to Silty Clay- Wet	▽ 20		6-SS	8				29		LL=47 PL=20 PI=27
		130	7-SS	12				25		
Grayish Brown Silty Clay - Very Moist to Wet	30		8-SS	7				32		LL=48 PL=22 PI=26
		120	9-SS	11				21		
	40		10-SS	15				23		
Dark Gray Clay - very Moist		110	11-SS	15				23		
Dark Brown fine to coarse Sandy Clay - Very Moist	50		12-SS	33				21		P_{200}=50%

Groundwater encountered at 21 feet
Boring Terminated at about 51.5 feet (EL. 106.4')

Water Observation Data	Remarks:
▽ Water Encountered During Drilling: 21 ft.	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
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.

GILES LOG REPORT 2G-1708013.GPJ GILES.GDT 12/18/17

TEST BORING LOG

BORING NO. & LOCATION: B- 2	PROPOSED PUBLIC STORAGE BUILDINGS	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 157.2 feet	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
COMPLETION DATE: 10/11/17		
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of asphaltic concrete over 3 inches of aggregate base										
Light Brown Silty Clay - Moist (Possible Fill)		155	1-SS	8				18		
Light Brown Silty Clay, some fine Sand - Very Moist (Native)	5		2-CS	29				19		
		150	3-CS	21				23		
Olive Brown Silty Clay, trace of fine Sand - very Moist	10		4-SS	5				25		
		145								
	15		5-SS	7				24		
		140								
	20 ▽		6-SS	8				31		
		135								
	25		7-SS	9				30		

Groundwater encountered at 20' 9" (20.75')
Boring Terminated at about 26.5 feet (EL. 130.7')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 20.75 ft.	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
	Cave Depth At End of Drilling:	
▼	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- 3	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 157.3 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/10/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 5 inches of aggregate base										
Dark Gray Clay with asphalt and brick pieces - Moist (Fill)			1-SS	26		4.5+		15		
	2.5	155.0								
Light Brown fine Sandy Clay - Moist (Native)			2-SS	18				13		P_{200}=94%
	5.0	152.5								

No groundwater encountered
Boring Terminated at about 5 feet (EL. 152.3')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After 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	TEST BORING LOG	
SURFACE ELEVATION: 158.8 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	GILES ENGINEERING ASSOCIATES, INC.
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 8 inches of aggregate base										
Light Brown Silty coarse Sand with some Gravel - Very Moist (Fill)			1-SS	11				20		
Light Brown Silty Clay, trace to little fine Sand - Moist to Very Moist (Native)		155	2-SS	11		4.25		19		
	5									
			3-SS	10		4.0		20		
		150								
	10		4-SS	8				20		
		145								
Light Gray to Brown Clay - Very Moist to Wet	15		5-SS	6				24		
		140								
	20		6-SS	10				25		
	▽									
		135								
	25		7-SS	9				23		

Groundwater encountered at 21.5 feet
Boring Terminated at about 26.5 feet (EL. 132.3')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 21.5 ft.	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	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- 5	TEST BORING LOG	
SURFACE ELEVATION: 158.1 feet	PROPOSED PUBLIC STORAGE BUILDINGS	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 5 inches of aggregate base										
Dark Brown Clay with some asphalt pieces - Moist (Fill)			1-SS	13		4.5+		18		
		155								
Light Brown Silty Clay, trace to little fine Sand - Very Moist (Native)			2-SS	8		3.75		20		
	5									
			3-CS	22				20		Dd=104.4 pcf
		150								
	10									
			4-CS	15				21		Dd=103.8 pcf
		145								
	15									
Olive Brown Silty Clay - Very Moist to Wet			5-SS	5				26		
		140								
	20 ▽									
			6-SS	9				23		
		135								
	25									
Light Grayish Brown fine Sandy Clay - Wet			7-SS	11				25		

Groundwater encountered at 20' 9" (20.75')
Boring Terminated at about 26.5 feet (EL. 131.6')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 20.75 ft.	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
	Cave Depth At End of Drilling:	
▼	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- 6	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 158.6 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/10/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 7 inches of aggregate base			1-SS	25		4.5+		16		
Dark Gray Clay with some asphalt pieces - Moist (Fill)			2-SS	8				18		
Light Brown fine Sandy Clay, some Silt - Moist to Very Moist (Native)			3-SS	10		3.0		19		
		150								
	10		4-SS	7				21		
Light Grayish Brown fine Sandy Clay to Silty Clay - Very Moist to Wet			5-SS	6				24		
		140								
	▽ 20		6-SS	6				28		LL=36 PL=22 PI=14
			7-SS	10				28		
		130								
Light Gray Clay - Very Moist to Wet	30		8-SS	7				30		LL=53 PL=28 PI=25
			9-SS	13				31		
		120								
Light Brown fine Sandy Clay - Very Moist	40		10-SS	10				24		P_{200}=61%
Light Brown Fine to coarse Sand with Gravel - Very Moist			11-SS	54				8		P_{200}=10%
		110								
Light Grayfine to coarse Sandy Clay, little Gravel - Very Moist	50		12-SS	11				15		LL=40 PL=24 PI=16

Groundwater encountered at 20' 8" (20.67')
Boring Terminated at about 51.5 feet (EL. 107.1')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 20.67 ft.	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	
	Cave Depth At End of Drilling:	
▼	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- 7	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 159.4 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 8 inches of aggregate base										
Dark Gray and Light Brown Sandy Clay with large asphalt and brick pieces - Moist (Fill)			1-SS	14		4.5+		18		EI=29 (low)
Light Brown Silty Clay, trace to little Sand - moist (Native)	5	155	2-SS	9		4.5+		17		
			3-SS	10		4.25		18		
	10	150	4-SS	8		3.75		20		
Olive Brown Silty Clay to fine Sandy Clay - Very Moist	15	145	5-SS	7				22		
	20	140	6-SS	8				27		
	25	135	7-SS	8				23		

No groundwater encountered
Boring Terminated at about 26.5 feet (EL. 132.9')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After 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- 8	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 159.1 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of asphaltic concrete over 4 inches of aggregate base			1-SS	13		4.5+		15		
Dark Brown Silty Clay, trace to little Sand - Moist (Possible Native)			2-SS	10				18		
Light Brown Silty Clay to fine Sandy Clay - Moist to Very Moist (Native)			3-SS	10		3.5		20		
	10	150	4-SS	9				20		
			5-SS	7				23		
Olive Brown to Light Brown Silty Clay to fine Sandy Clay - Very Moist	20	140	6-SS	7				22		
			7-SS	6				25		

No groundwater encountered
Boring Terminated at about 26.5 feet (EL. 132.6')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After Drilling:	
Cave Depth After Drilling:	

GILES LOG REPORT 2G-1708013.GPJ GILES.GDT 12/13/17

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

BORING NO. & LOCATION: B- 9	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 158.3 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 5 inches of asphaltic concrete over 2 inches of aggregate base										
Dark Brown Clay with some asphalt pieces - Moist (Fill)		157.5	1-SS	9			4.5	18		
	2.5	155.0								
Light Brown Silty Clay, trace to little Sand - Moist (Native)			2-SS	17				17		P_{200}=84%
	5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 153.3')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After 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-10	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 157.4 feet	PROPOSED PUBLIC STORAGE BUILDINGS	
COMPLETION DATE: 10/11/17	NWC W. CAPITOL EXPRESSWAY & SNELL AVENUE SAN JOSE, CA	
FIELD REP: MONICA SELL	PROJECT NO: 2G-1708013	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 3 inches of aggregate base										
Dark Brown Silty Clay - Moist (Possible Native)			1-SS	15		4.5+		16		
	2.5	155.0								
Light Brown Silty Clay, trace to little fineSand - Moist (Native)			2-SS	13				18		P_{200}=93%
	5.0	152.5								

No groundwater encountered
Boring Terminated at about 5 feet (EL. 152.4')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	
Cave Depth At End of Drilling:	
Water Level After 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.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-1	PROJECT: Proposed Four-Story Public Storage Building	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 159.0'	PROJECT LOCATION: NWC W. Capitol Expressway & Snell Ave.	Milwaukee Atlanta
COMPLETION DATE: 9/25/13	San Jose, CA	Dallas Wasington, D.C.
FIELD REPRESENTATIVE: Larry Ballard	GILES PROJECT NUMBER: 2G-1309003	Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 3 inches of aggregate base		1 SS	11	3.7			19		LL = 43 PL = 20 PI = 23
Dark Brown Sandy Clay - Moist (Fill)									
Light Brown Clay - Moist (Native)	5	2 SS	5				19		
Light Brown Clay - Moist		3 SS	14		4.5+		18		
	10	4 SS	5		1.5		19		
	15	5 SS	4		1.5		22		
	20	6 SS	6				27		LL = 47 PL = 18 (PI=29) P_{200} = 96%
Light Brown to Olive Brown Silty fine Sand to Light Brown Silty Clay - Moist	25	7 SS	8				22		P_{200} = 58%
Olive Brown Clay - Very Moist to Wet	30	8 SS	8				32		
	35	9 SS	7				32		LL = 48 PL = 22 (PI = 26)
	⊻ 40	10 SS	4				28		
Olive Brown to Dark Gray Silty fine Sand - Very Moist	45	11 SS	11				30		P_{200} = 49%

Boring terminated at 46.5 feet.
Groundwater encountered at 40 feet.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 40	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	qu = Unconfined compressive strength
WATER LEVEL AFTER HOURS:	qp = Pocket penetrometer
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

BORING NO. & LOCATION: B-2	PROJECT: Proposed Four-Story Public Storage Building
SURFACE ELEVATION: 158.5'	PROJECT LOCATION: NWC W. Capitol Expressway & Snell Ave.
COMPLETION DATE: 9/25/13	San Jose, CA
FIELD REPRESENTATIVE: Larry Ballard	GILES PROJECT NUMBER: 2G-1309003

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphalic concrete over 3 inches of aggregate base.									
Dark Brown fine Sandy Clay - Moist (Fill)		1 SS	11	3.9			18		
Light Brown Clay - Moist (Native)		2 CS	11		4.5+		19		Dd = 108.6 pcf
	5								
Light Brown Clay - Moist to Very Moist		3 SS	9		2.75		20		
	10	4 SS	7		2.0		19		
	15	5 SS	5		1.25		26		
	20	6 SS	7				31		

Boring terminated at 21.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	qp = Pocket penetrometer
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION:	PROJECT:
B-3	Proposed Four-Story Public Storage Building
SURFACE ELEVATION:	PROJECT LOCATION:
159.0'	NWC W. Capitol Expressway & Snell Ave.
COMPLETION DATE:	
9/25/13	San Jose, CA
FIELD REPRESENTATIVE:	
Larry Ballard	GILES PROJECT NUMBER: 2G-1309003

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 4.5 inches of aggregate base									
Olive Light Brown to Dark Brown fine Sandy Clay - Moist (Fill)		1 SS	17				15		
Light Brown fine Sandy Clay - Moist (Possible Native)		2 SS	7		2.75		14		
	5								
		3 CS	26		4.5+		15		Dd = 108.8 pcf
Light Brown Clay - Moist	10	4 SS	11		3.5		17		
Light Brown Clay - Very Moist	15	5 SS	5				22		

Boring terminated at 16.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	qp = Pocket penetrometer
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

BORING NO. & LOCATION: B-4	PROJECT: Proposed Four-Story Public Storage Building
SURFACE ELEVATION: 160.5'	PROJECT LOCATION: NWC W. Capitol Expressway & Snell Ave.
COMPLETION DATE: 9/25/13	San Jose, CA
FIELD REPRESENTATIVE: Larry Ballard	GILES PROJECT NUMBER: 2G-1309003

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Approximately 2 inches of asphaltic concrete over 3 inches of aggregate base									
Olive Brown fine and coarse Sandy Clay - Moist (Fill)		1 SS	12				15		
Light Brown fine Sandy Clay - Moist (Native)		2 CS	18		3.0		20		Dd = 103.5 pcf
	5								
Dark Brown Clay - Moist to Very Moist		3 SS	13		4.5+		17		
	10	4 SS	6		2.75		20		
	15	5 SS	5		1.75		22		

Boring terminated at 16.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	qp = Pocket penetrometer
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

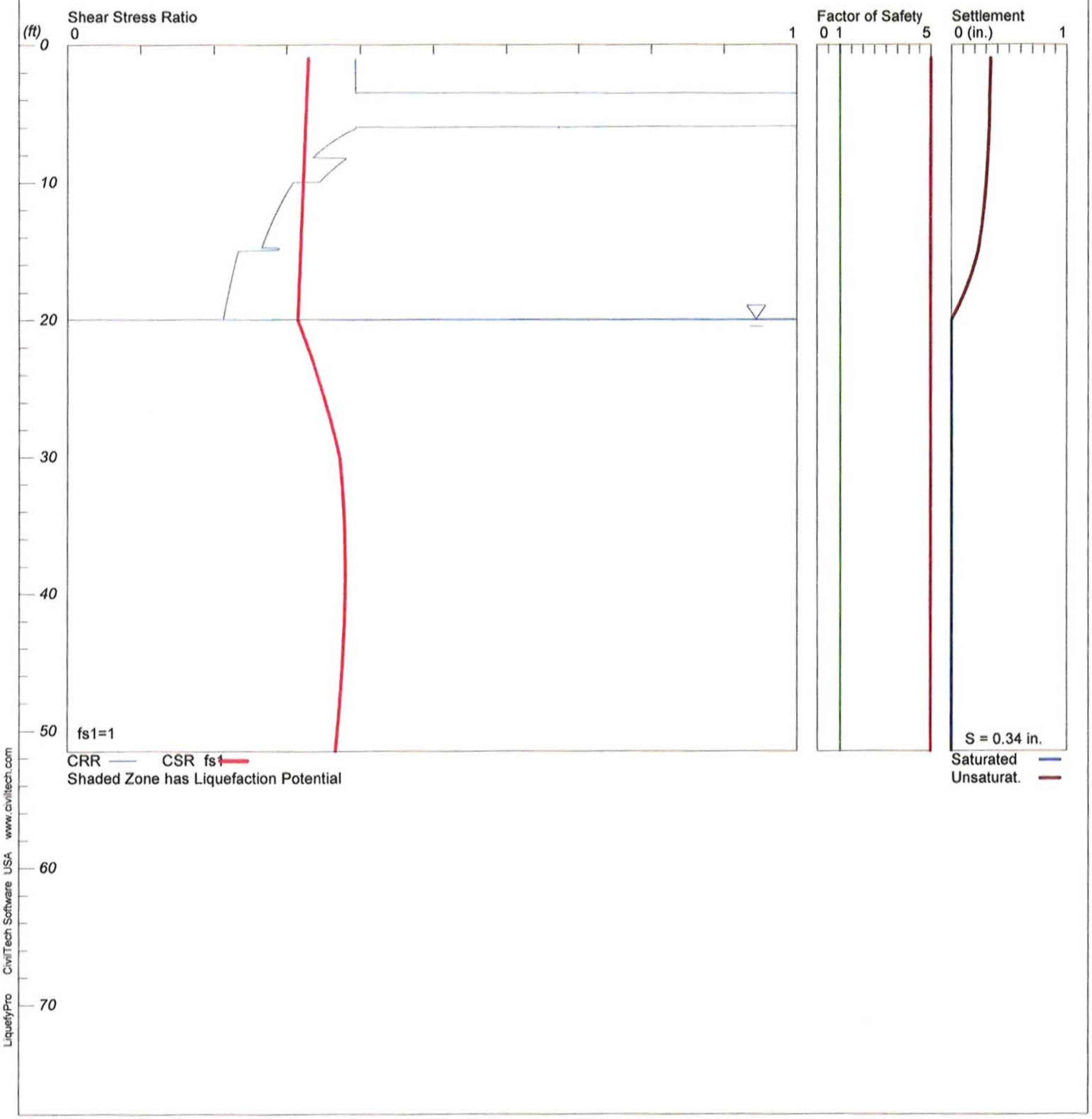
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between borings. Location of Test Boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

Public Storage Capitol Expressway, San Jose, CA

Hole No.=B-1   Water Depth=20 ft

Magnitude=6.69
Acceleration=0.508g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\Edgar Gatus\2G-1708013, PS Capitol Expressway, San Jose\B-1.liq
Title: Public Storage Capitol Expressway, San Jose, CA
Subtitle: 2G-1708013

Surface Elev.=
Hole No.=B-1
Depth of Hole= 51.50 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.51 g
Earthquake Magnitude= 6.69

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=51.50 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.51 g
Earthquake Magnitude=6.69
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu/Seed
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=1)
10. Use Curve Smoothing: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
1.00	9.00	120.00	90.00
3.50	14.00	120.00	90.00
6.00	9.00	120.00	90.00
10.00	8.00	120.00	88.00
15.00	6.00	120.00	88.00
20.00	8.00	120.00	NoLiq
25.00	12.00	120.00	NoLiq
30.00	7.00	120.00	NoLiq
35.00	11.00	120.00	NoLiq

40.00	15.00	120.00	NoLiq
45.00	15.00	120.00	NoLiq
50.00	33.00	120.00	50.00

Output Results:
Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.34 in.
Total Settlement of Saturated and Unsaturated Sands=0.34 in.
Differential Settlement=0.170 to 0.224 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
1.00	0.39	0.33	5.00	0.00	0.34	0.34
1.50	0.39	0.33	5.00	0.00	0.34	0.34
2.00	0.39	0.33	5.00	0.00	0.34	0.34
2.50	0.39	0.33	5.00	0.00	0.34	0.34
3.00	0.39	0.33	5.00	0.00	0.33	0.33
3.50	0.39	0.33	5.00	0.00	0.33	0.33
4.00	2.68	0.33	5.00	0.00	0.33	0.33
4.50	2.68	0.33	5.00	0.00	0.33	0.33
5.00	2.68	0.33	5.00	0.00	0.33	0.33
5.50	2.68	0.33	5.00	0.00	0.33	0.33
6.00	0.39	0.33	5.00	0.00	0.33	0.33
6.50	0.38	0.33	5.00	0.00	0.32	0.32
7.00	0.36	0.32	5.00	0.00	0.32	0.32
7.50	0.35	0.32	5.00	0.00	0.32	0.32
8.00	0.34	0.32	5.00	0.00	0.32	0.32
8.50	0.38	0.32	5.00	0.00	0.31	0.31
9.00	0.36	0.32	5.00	0.00	0.31	0.31
9.50	0.35	0.32	5.00	0.00	0.31	0.31
10.00	0.31	0.32	5.00	0.00	0.30	0.30
10.50	0.30	0.32	5.00	0.00	0.30	0.30
11.00	0.30	0.32	5.00	0.00	0.29	0.29
11.50	0.29	0.32	5.00	0.00	0.29	0.29
12.00	0.29	0.32	5.00	0.00	0.28	0.28
12.50	0.28	0.32	5.00	0.00	0.27	0.27
13.00	0.28	0.32	5.00	0.00	0.27	0.27
13.50	0.27	0.32	5.00	0.00	0.26	0.26
14.00	0.27	0.32	5.00	0.00	0.25	0.25
14.50	0.27	0.32	5.00	0.00	0.24	0.24
15.00	0.23	0.32	5.00	0.00	0.23	0.23
15.50	0.23	0.32	5.00	0.00	0.21	0.21
16.00	0.23	0.32	5.00	0.00	0.20	0.20
16.50	0.23	0.32	5.00	0.00	0.18	0.18
17.00	0.22	0.32	5.00	0.00	0.16	0.16
17.50	0.22	0.32	5.00	0.00	0.14	0.14
18.00	0.22	0.32	5.00	0.00	0.12	0.12
18.50	0.22	0.32	5.00	0.00	0.09	0.09
19.00	0.22	0.32	5.00	0.00	0.06	0.06
19.50	0.21	0.32	5.00	0.00	0.03	0.03
20.00	0.21	0.31	5.00	0.00	0.00	0.00
20.50	2.00	0.32	5.00	0.00	0.00	0.00
21.00	2.00	0.32	5.00	0.00	0.00	0.00
21.50	2.00	0.33	5.00	0.00	0.00	0.00
22.00	2.00	0.33	5.00	0.00	0.00	0.00
22.50	2.00	0.33	5.00	0.00	0.00	0.00
23.00	2.00	0.33	5.00	0.00	0.00	0.00
23.50	2.00	0.34	5.00	0.00	0.00	0.00
24.00	2.00	0.34	5.00	0.00	0.00	0.00
24.50	2.00	0.34	5.00	0.00	0.00	0.00
25.00	2.00	0.35	5.00	0.00	0.00	0.00

25.50	2.00	0.35	5.00	0.00	0.00	0.00
26.00	2.00	0.35	5.00	0.00	0.00	0.00
26.50	2.00	0.35	5.00	0.00	0.00	0.00
27.00	2.00	0.36	5.00	0.00	0.00	0.00
27.50	2.00	0.36	5.00	0.00	0.00	0.00
28.00	2.00	0.36	5.00	0.00	0.00	0.00
28.50	2.00	0.36	5.00	0.00	0.00	0.00
29.00	2.00	0.37	5.00	0.00	0.00	0.00
29.50	2.00	0.37	5.00	0.00	0.00	0.00
30.00	2.00	0.37	5.00	0.00	0.00	0.00
30.50	2.00	0.37	5.00	0.00	0.00	0.00
31.00	2.00	0.37	5.00	0.00	0.00	0.00
31.50	2.00	0.37	5.00	0.00	0.00	0.00
32.00	2.00	0.37	5.00	0.00	0.00	0.00
32.50	2.00	0.38	5.00	0.00	0.00	0.00
33.00	2.00	0.38	5.00	0.00	0.00	0.00
33.50	2.00	0.38	5.00	0.00	0.00	0.00
34.00	2.00	0.38	5.00	0.00	0.00	0.00
34.50	2.00	0.38	5.00	0.00	0.00	0.00
35.00	2.00	0.38	5.00	0.00	0.00	0.00
35.50	2.00	0.38	5.00	0.00	0.00	0.00
36.00	2.00	0.38	5.00	0.00	0.00	0.00
36.50	2.00	0.38	5.00	0.00	0.00	0.00
37.00	2.00	0.38	5.00	0.00	0.00	0.00
37.50	2.00	0.38	5.00	0.00	0.00	0.00
38.00	2.00	0.38	5.00	0.00	0.00	0.00
38.50	2.00	0.38	5.00	0.00	0.00	0.00
39.00	2.00	0.38	5.00	0.00	0.00	0.00
39.50	2.00	0.38	5.00	0.00	0.00	0.00
40.00	2.00	0.38	5.00	0.00	0.00	0.00
40.50	2.00	0.38	5.00	0.00	0.00	0.00
41.00	2.00	0.38	5.00	0.00	0.00	0.00
41.50	2.00	0.38	5.00	0.00	0.00	0.00
42.00	2.00	0.38	5.00	0.00	0.00	0.00
42.50	2.00	0.38	5.00	0.00	0.00	0.00
43.00	2.00	0.38	5.00	0.00	0.00	0.00
43.50	2.00	0.38	5.00	0.00	0.00	0.00
44.00	2.00	0.38	5.00	0.00	0.00	0.00
44.50	2.00	0.38	5.00	0.00	0.00	0.00
45.00	2.00	0.37	5.00	0.00	0.00	0.00
45.50	2.00	0.37	5.00	0.00	0.00	0.00
46.00	2.00	0.37	5.00	0.00	0.00	0.00
46.50	2.00	0.37	5.00	0.00	0.00	0.00
47.00	2.00	0.37	5.00	0.00	0.00	0.00
47.50	2.00	0.37	5.00	0.00	0.00	0.00
48.00	2.00	0.37	5.00	0.00	0.00	0.00
48.50	2.00	0.37	5.00	0.00	0.00	0.00
49.00	2.00	0.37	5.00	0.00	0.00	0.00
49.50	2.00	0.37	5.00	0.00	0.00	0.00
50.00	2.00	0.37	5.00	0.00	0.00	0.00
50.50	2.58	0.37	5.00	0.00	0.00	0.00
51.00	2.58	0.37	5.00	0.00	0.00	0.00
51.50	2.58	0.37	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: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils

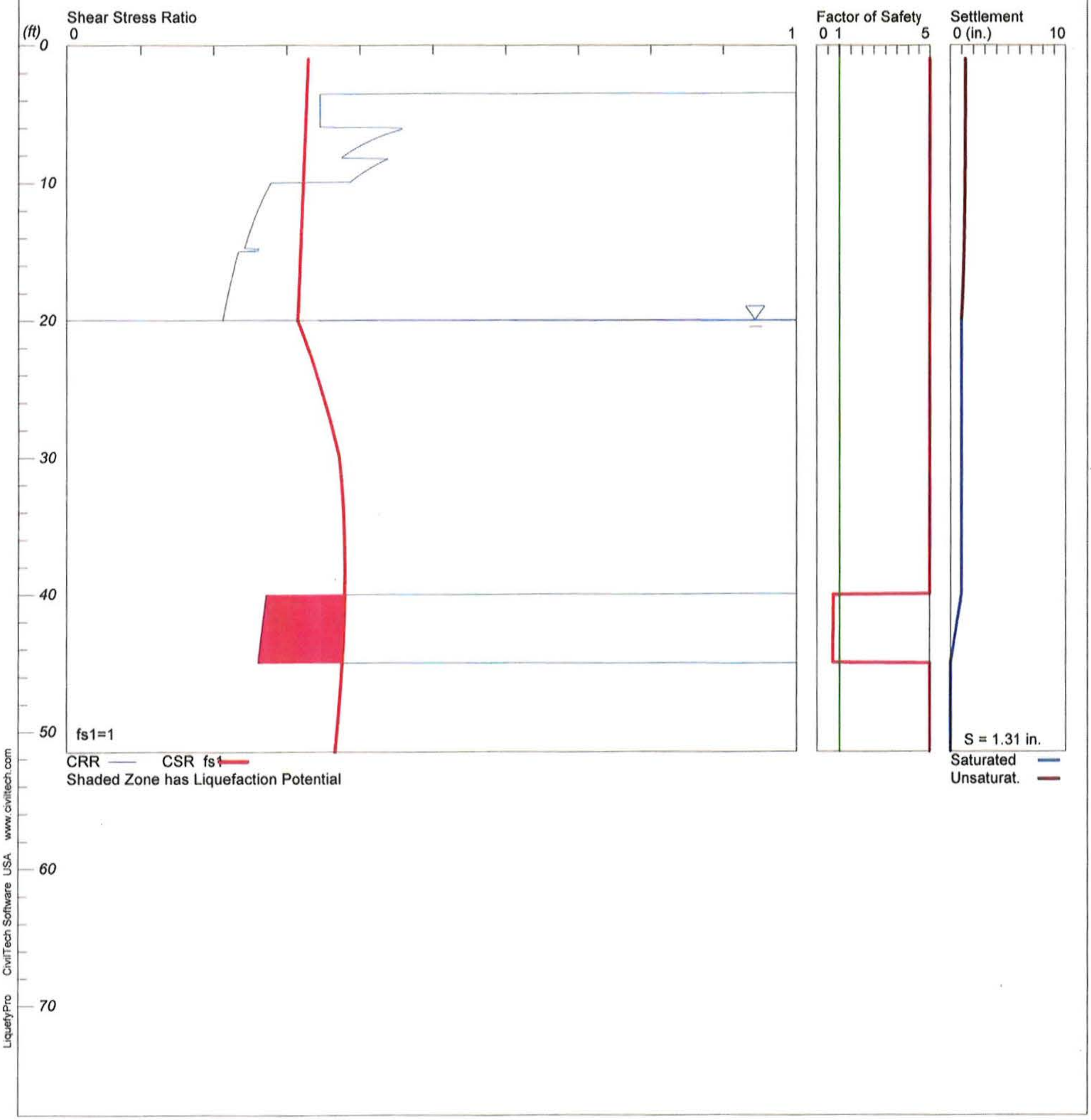
```
        CSRsf               Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)
        F.S.                Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat               Settlement from saturated sands
        S_dry               Settlement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

LIQUEFACTION ANALYSIS

Public Storage Capitol Expressway, San Jose, CA

Hole No.=B-6   Water Depth=20 ft

Magnitude=6.69
Acceleration=0.508g



CivilTech Corporation   **2G-1708013**   **Plate A-1**

****
LIQUEFACTION ANALYSIS SUMMARY

****

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Input File Name: P:\Edgar Gatus\2G-1708013, PS Capitol Expressway, San Jose\B-6.liq
Title: Public Storage Capitol Expressway, San Jose, CA
Subtitle: 2G-1708013

Surface Elev.=
Hole No.=B-6
Depth of Hole= 51.50 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration= 0.51 g
Earthquake Magnitude= 6.69

Input Data:
Surface Elev.=
Hole No.=B-6
Depth of Hole=51.50 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.51 g
Earthquake Magnitude=6.69
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu/Seed
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=1)
10. Use Curve Smoothing: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
1.00	25.00	120.00	90.00
3.50	8.00	120.00	90.00
6.00	10.00	120.00	90.00
10.00	7.00	120.00	90.00
15.00	6.00	120.00	90.00
20.00	6.00	120.00	NoLiq
25.00	10.00	120.00	NoLiq
30.00	7.00	120.00	NoLiq
35.00	13.00	120.00	NoLiq

40.00	10.00	120.00	61.00
45.00	54.00	120.00	10.00
50.00	11.00	120.00	NoLiq

Output Results:
Settlement of Saturated Sands=0.96 in.
Settlement of Unsaturated Sands=0.36 in.
Total Settlement of Saturated and Unsaturated Sands=1.31 in.
Differential Settlement=0.657 to 0.868 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
1.00	2.68	0.33	5.00	0.96	0.36	1.31
1.50	2.68	0.33	5.00	0.96	0.36	1.31
2.00	2.68	0.33	5.00	0.96	0.36	1.31
2.50	2.68	0.33	5.00	0.96	0.35	1.31
3.00	2.68	0.33	5.00	0.96	0.35	1.31
3.50	2.68	0.33	5.00	0.96	0.35	1.31
4.00	0.35	0.33	5.00	0.96	0.35	1.31
4.50	0.35	0.33	5.00	0.96	0.35	1.31
5.00	0.35	0.33	5.00	0.96	0.35	1.31
5.50	0.35	0.33	5.00	0.96	0.34	1.30
6.00	0.46	0.33	5.00	0.96	0.34	1.30
6.50	0.43	0.33	5.00	0.96	0.34	1.30
7.00	0.41	0.32	5.00	0.96	0.34	1.30
7.50	0.39	0.32	5.00	0.96	0.33	1.29
8.00	0.38	0.32	5.00	0.96	0.33	1.29
8.50	0.43	0.32	5.00	0.96	0.33	1.29
9.00	0.41	0.32	5.00	0.96	0.33	1.28
9.50	0.40	0.32	5.00	0.96	0.32	1.28
10.00	0.28	0.32	5.00	0.96	0.32	1.28
10.50	0.27	0.32	5.00	0.96	0.31	1.27
11.00	0.27	0.32	5.00	0.96	0.31	1.27
11.50	0.26	0.32	5.00	0.96	0.30	1.26
12.00	0.26	0.32	5.00	0.96	0.29	1.25
12.50	0.26	0.32	5.00	0.96	0.28	1.24
13.00	0.25	0.32	5.00	0.96	0.28	1.23
13.50	0.25	0.32	5.00	0.96	0.26	1.22
14.00	0.25	0.32	5.00	0.96	0.25	1.21
14.50	0.24	0.32	5.00	0.96	0.24	1.20
15.00	0.23	0.32	5.00	0.96	0.23	1.19
15.50	0.23	0.32	5.00	0.96	0.21	1.17
16.00	0.23	0.32	5.00	0.96	0.20	1.16
16.50	0.23	0.32	5.00	0.96	0.18	1.14
17.00	0.22	0.32	5.00	0.96	0.16	1.12
17.50	0.22	0.32	5.00	0.96	0.14	1.10
18.00	0.22	0.32	5.00	0.96	0.12	1.07
18.50	0.22	0.32	5.00	0.96	0.09	1.05
19.00	0.22	0.32	5.00	0.96	0.06	1.02
19.50	0.21	0.32	5.00	0.96	0.03	0.99
20.00	0.21	0.31	5.00	0.96	0.00	0.96
20.50	2.00	0.32	5.00	0.96	0.00	0.96
21.00	2.00	0.32	5.00	0.96	0.00	0.96
21.50	2.00	0.33	5.00	0.96	0.00	0.96
22.00	2.00	0.33	5.00	0.96	0.00	0.96
22.50	2.00	0.33	5.00	0.96	0.00	0.96
23.00	2.00	0.33	5.00	0.96	0.00	0.96
23.50	2.00	0.34	5.00	0.96	0.00	0.96
24.00	2.00	0.34	5.00	0.96	0.00	0.96
24.50	2.00	0.34	5.00	0.96	0.00	0.96
25.00	2.00	0.35	5.00	0.96	0.00	0.96

25.50	2.00	0.35	5.00	0.96	0.00	0.96
26.00	2.00	0.35	5.00	0.96	0.00	0.96
26.50	2.00	0.35	5.00	0.96	0.00	0.96
27.00	2.00	0.36	5.00	0.96	0.00	0.96
27.50	2.00	0.36	5.00	0.96	0.00	0.96
28.00	2.00	0.36	5.00	0.96	0.00	0.96
28.50	2.00	0.36	5.00	0.96	0.00	0.96
29.00	2.00	0.37	5.00	0.96	0.00	0.96
29.50	2.00	0.37	5.00	0.96	0.00	0.96
30.00	2.00	0.37	5.00	0.96	0.00	0.96
30.50	2.00	0.37	5.00	0.96	0.00	0.96
31.00	2.00	0.37	5.00	0.96	0.00	0.96
31.50	2.00	0.37	5.00	0.96	0.00	0.96
32.00	2.00	0.37	5.00	0.96	0.00	0.96
32.50	2.00	0.38	5.00	0.96	0.00	0.96
33.00	2.00	0.38	5.00	0.96	0.00	0.96
33.50	2.00	0.38	5.00	0.96	0.00	0.96
34.00	2.00	0.38	5.00	0.96	0.00	0.96
34.50	2.00	0.38	5.00	0.96	0.00	0.96
35.00	2.00	0.38	5.00	0.96	0.00	0.96
35.50	2.00	0.38	5.00	0.96	0.00	0.96
36.00	2.00	0.38	5.00	0.96	0.00	0.96
36.50	2.00	0.38	5.00	0.96	0.00	0.96
37.00	2.00	0.38	5.00	0.96	0.00	0.96
37.50	2.00	0.38	5.00	0.96	0.00	0.96
38.00	2.00	0.38	5.00	0.96	0.00	0.96
38.50	2.00	0.38	5.00	0.96	0.00	0.96
39.00	2.00	0.38	5.00	0.96	0.00	0.96
39.50	2.00	0.38	5.00	0.96	0.00	0.96
40.00	2.00	0.38	5.00	0.96	0.00	0.96
40.50	0.27	0.38	0.71*	0.87	0.00	0.87
41.00	0.27	0.38	0.71*	0.78	0.00	0.78
41.50	0.27	0.38	0.71*	0.68	0.00	0.68
42.00	0.27	0.38	0.71*	0.59	0.00	0.59
42.50	0.26	0.38	0.70*	0.49	0.00	0.49
43.00	0.26	0.38	0.70*	0.40	0.00	0.40
43.50	0.26	0.38	0.70*	0.30	0.00	0.30
44.00	0.26	0.38	0.70*	0.20	0.00	0.20
44.50	0.26	0.38	0.69*	0.11	0.00	0.11
45.00	0.26	0.37	0.69*	0.01	0.00	0.01
45.50	2.62	0.37	5.00	0.00	0.00	0.00
46.00	2.61	0.37	5.00	0.00	0.00	0.00
46.50	2.61	0.37	5.00	0.00	0.00	0.00
47.00	2.61	0.37	5.00	0.00	0.00	0.00
47.50	2.60	0.37	5.00	0.00	0.00	0.00
48.00	2.60	0.37	5.00	0.00	0.00	0.00
48.50	2.60	0.37	5.00	0.00	0.00	0.00
49.00	2.59	0.37	5.00	0.00	0.00	0.00
49.50	2.59	0.37	5.00	0.00	0.00	0.00
50.00	2.59	0.37	5.00	0.00	0.00	0.00
50.50	2.00	0.37	5.00	0.00	0.00	0.00
51.00	2.00	0.37	5.00	0.00	0.00	0.00
51.50	2.00	0.37	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: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)
CRRm Cyclic resistance ratio from soils

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

APPENDIX B

FIELD PROCEDURES

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

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

GENERAL FIELD PROCEDURES

Test Boring Elevations

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

Test Boring Locations

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

Water Level Measurement

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

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

Borehole Backfilling Procedures

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

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

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

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

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

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

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

Bulk Sample (BS)

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

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

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

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

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

Sampling and Testing Procedures

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

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

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

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

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

Moisture Content (w) (ASTM D 2216)

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

Unconfined Compressive Strength (qu) (ASTM D 2166)

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

Calibrated Penetrometer Resistance (qp)

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

Vane-Shear Strength (qs)

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

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

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

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

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

Consolidation Test (ASTM D 2435)

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

Classification of Samples

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

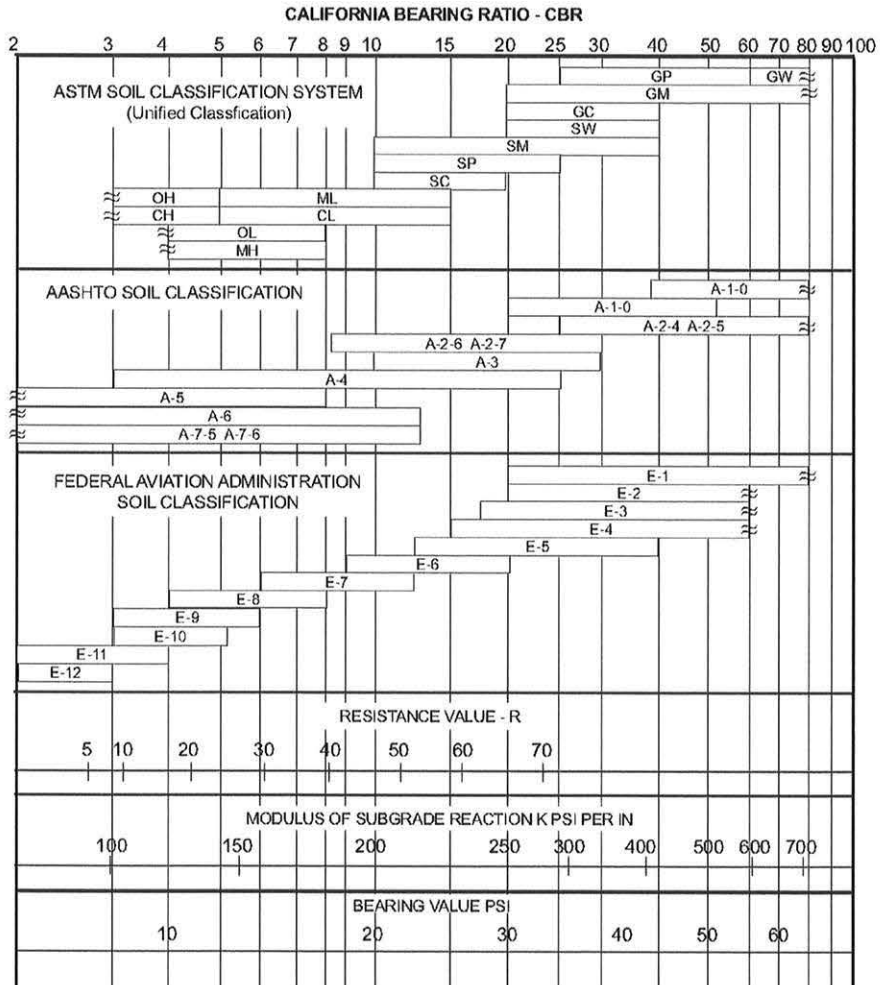
Laboratory Testing

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

California Bearing Ratio (CBR) Test ASTM D-1833

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

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



APPENDIX D

GENERAL INFORMATION

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

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

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

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

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

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

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

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

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

GENERAL COMMENTS

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

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

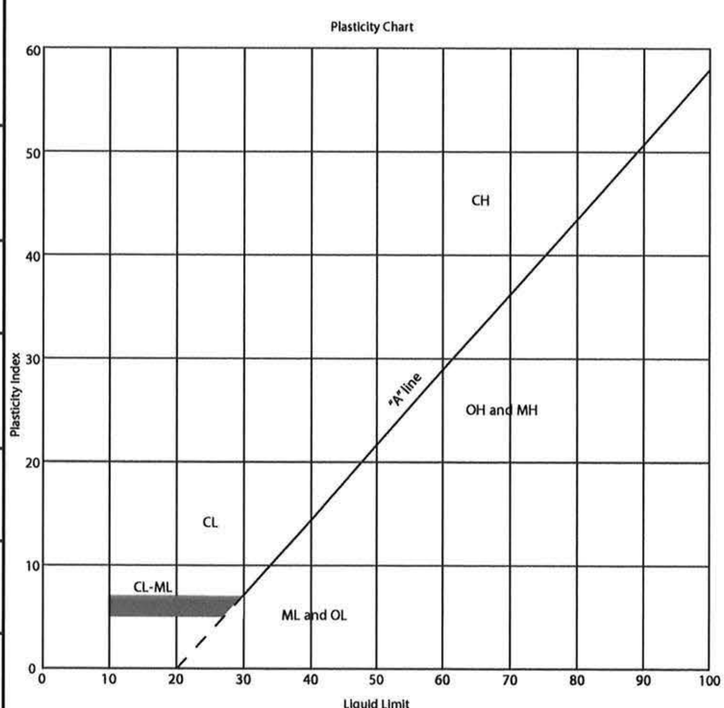
This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these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

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

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

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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



[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

Giles Engineering Associates, Inc.

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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