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Geotechnical Engineering Exploration and Analysis

Proposed Public Storage and Apartment Redevelopment
1020 and 1040 Terra Bella Avenue
Mountain View, California

Prepared for:

Public Storage
Glendale, California

September 17, 2021
Project No. 2G-2102004

GILES
ENGINEERING ASSOCIATES, INC.

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

- Atlanta, GA
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- Milwaukee, WI

September 17, 2021

Public Storage, Inc.
701 Western Avenue
Glendale, California 91201

Attention: Mr. Mark Wix
Vice President, Construction

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Public Storage and Apartment Redevelopment
1020 and 1040 Terra Bella Avenue
Mountain View, California
Giles Project No. 2G-2102004

Dear Mr. Wix:

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, please do not hesitate to call at any time.

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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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

- A Site Classification D is recommended for this site based upon the mapped geological features of the site also verified by test borings.
- According to the published provided online by *United States Geological Survey for the Palo Alto and Mountain View Quadrangles,* the western portion (about ¾) of the project is mapped as being underlain by silty clay and organic clay, fossiliferous; and the eastern portion (about ¼) of the project is mapped as alluvial sand, fine grained silt, and clay.
- Fill and possible fill soils were encountered at all boring and test pit locations, to depths ranging from approximately 6 to 10 feet below the existing grades. The fill/possible fill soils were generally moist to very moist, medium stiff to very stiff silty clay.
- Native soil encountered beneath the fill/possible fill consisted of moist to wet, soft to stiff silty clay with trace organics, and clayey silt, with trace sand and gravel; and moist to wet, loose to dense silty and clayey sands and gravels.
- Groundwater was encountered within the borings at depths ranging from about 7 to 8 feet below grade (El. 25 to 28 feet). The groundwater conditions encountered is presented in the following table

Site Development

- This project will involve the demolition of existing one-story buildings and constructing two, 5-story Public Storage buildings and a 7-story apartment building near existing grade.

Building Foundations

- The proposed structure may be supported by a shallow foundation system (grade beam with slab-on-grade, or mat/slab if desired) constructed on the existing soil improved with a ground improvement system.

Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 6 and 8 inches (parking stall and drive lane areas, respectively) of aggregate base course, over 12 inches of compacted subgrade.
- Portland Cement Concrete: 6 inches in thickness in high stress areas such as entrance/exit aprons lane and in trash enclosure loading zone with a 4 inch granular base.

Construction Consideration

- Below grade construction difficulties are expected due to the water table.

- Excavations as described in our report should not be performed near Caltrans approaches, abutments, and columns near the north property line and extending into the northern area of the Public Storage property. If vibratory installation methods are used, existing structural features and buildings near the site should be surveyed ad monitored for pre-existing damages.

RED – Giles has given the project a Red designation because of significant and extensive geotechnical-related concerns consisting of compressible clay deposits at the site requiring a ground improvement system with conventional shallow foundation.

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 redevelopment. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report.

Geotechnical-related recommendations for design and construction of the foundation, ground-bearing floor slab for the proposed building, pavement, and retaining walls are provided in this report. 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 project consists of two parcels (‘project’) located at 1020 and 1040 Terra Bella Avenue in the city of Mountain View, California. The project is located in Santa Clara County and currently the western parcel is developed with several single-story Public Storage buildings, an office building, and a Caltrans retaining wall at the northwestern area of the site along the property line; the eastern portion of the project, 1020 Terra Bella Avenue, is currently vacant and partially paved.

The project is bordered on the north by a Caltrans approach, abutment, and columns for the Highway 101 entrance ramp and bridge about 33 feet in height, with the approach ranging from about 0 to 33 feet in height. To the west of the sites are commercial buildings, to the south is Terra Bella Avenue, and to the east is San Rafael Avenue.

The project is situated at approximately latitude 37.4092° North, longitude -122.0736° West.

A topographic survey prepared by BKF Engineers dated August 28, 2021, indicates elevations at the site range from approximately El. 33 ft. at the northwest corner of the site to about El. 36 ft. at the southeastern corner of the property.

2.2 Proposed Project Description

Based on information provided, two, at-or-near grade, 5-story Public Storage structures (Building 1 about 40,000 sq.ft. footprint and Building 2 about 49,711 sq.ft footprint) are planned at the western and eastern areas of the development; the southeastern area of the project is planned for an at-or-near grade 7-story apartment building (total building footprint of about 45,180 sq.ft and gross building footprint of 30,497 sq.ft.), with the two lower levels (Levels 1 and 2) consisting of parking, open space, and few apartments, with five levels of apartments above the parking levels. The restructuring of the parcels, and layout of the structures is shown on Figure 1. The existing asphaltic concrete pavements, retaining walls, and single-story buildings at the site were visually observed to be in good condition.

Based on information provided by the structural engineer, the dead and live loads for the 7-story apartment structure are 850 psf DL plus 110 psf LL. Although specific structural design for this building at the project site is not finalized the lower level parking for the building is planned to be constructed of concrete (podium) with 27 foot column spans with wood-framed residential units above the parking structure.

The dead and live loads for the two, 5-story public storage buildings are 625 psf DL + 250 psf LL. The buildings will be supported by perimeter load-bearing walls and interior columns. The building is expected to be framed with interior columns on a 10 foot by 10 foot grid. The columns will have a maximum axial load of 100 kips. The maximum combined live and dead load supported by the perimeter bearing walls is estimated to be 5 kips per lineal foot. The live load supported by the ground floor slab is anticipated to be a maximum of 125 pounds per square foot (psf). Other planned site improvements include minor retaining walls, concrete walkways, and new pavement.

Topographical information indicates the site is relatively level within each planned structure area, and therefore the planned lower floor elevations for the proposed new structures will be within 1 to 2 feet of the existing grades. Therefore, site grading will require about ½ to 1 foot of fill within each storage or apartment structure to establish pad subgrade elevations.

Parking stalls and drive lanes will be constructed generally within the storage area of the project. Parking stalls and drive pavement areas are expected to be subjected to passenger vehicle traffic and large moving trucks. The parking stalls and drive lanes are anticipated to be subjected to a daily traffic loading of 1 to 2 heavy trucks per day (5 Equivalent Single/Axle Loads) and pavement design is based on a 20 year pavement design life.

2.3 Background Information

A copy of the geotechnical report prepared by Kleinfelder (referenced) for the Highway 101 approach, abutment, and bridge columns was obtained on September 6, 2021 and reviewed. Based upon our review of their subsurface data, laboratory data, and calculations regarding settlement, it is determined that site soils and expected settlements at the project site are similar to those at the northern Caltrans property considering the different construction features.

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration consisted of drilling sixteen (16) test borings (Storage Building 1: B-1 through B-6; Storage Building 2: B-12 to B-18; Apartment Building: B-7 to B-9;) and two (2) test pits (TP-1 and TP-2) along the northern existing drive isle. The test borings and test pits were completed between April 6 and April 14, 2021. The approximate test boring locations are shown on the Test Boring and Test Pit Location Plan (Figure 1).

The Test Boring and Test Pit Location Plan and Test Boring Logs (Records of Subsurface Exploration), liquefaction and laboratory test results are enclosed in Appendix A. Field and laboratory test procedures are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

3.2 Subsurface Conditions

The subsurface conditions as subsequently described have been simplified somewhat for ease of report interpretation. A more detailed description of the subsurface conditions at the test boring locations is provided by the test boring logs enclosed in Appendix A of this report. In addition to Giles subsurface exploration and laboratory data, additional information was reviewed including logs of borings near this site from previous studies and supplemental laboratory data by others as described in Section 2.3.

Geologic Setting

According to the published geological maps provided online by *United States Geological Survey for the Palo Alto and Mountain View Quadrangles,* the western portion (about ¾) of the project is mapped as being underlain by silty clay and organic clay, fossiliferous; and the eastern portion (about ¼) of the project is mapped as alluvial sand, fine grained silt, and clay.

Based on the soils encountered within the test borings, it appears that the geologic contacts between the two units has occurred over significant geological time with inconsistent layers (thickness and lateral extents) of deposited gravels, sands, silts and clays.

Pavement

Existing asphaltic concrete (AC) pavements approximately 2 to 5 inches thick with 3 to 12 inches of aggregate base material were encountered at the surface at the test boring locations. Based on our visual examination, the existing pavements were observed to be in good condition.

Fill/Possible Fill

Fill and possible fill soils were encountered at all boring and test pit locations, to depths ranging from approximately 6 to 10 feet below the existing grades. The fill/possible fill soils were generally moist to very moist, medium stiff to very stiff silty clay.

Native Soil

Native soil encountered beneath the fill/possible fill consisted of moist to wet, soft to stiff silty clay with trace organics, and clayey silt, with trace sand and gravel; and moist to wet, loose to dense silty and clayey sands and gravels.

Groundwater

Groundwater was encountered within the borings at depths ranging from about 7 to 8 feet below grade (El. 25 to 28 feet). The groundwater conditions encountered is presented in the following table.

TABLE 1

Groundwater Elevation Measurements

Test Boring / Test Pit	Date Recorded	Approx. Surface Elevation (ft)	Depth to Water (ft)	Approx. Water Elevation (feet)
Public Storage Building 1				
B-1	4/6/21	32	7	El. 25
B-2	4/6/21	33	7	El. 26
B-3	4/6/21	32	7	El. 25
B-4	4/6/21	33	8	El. 25
B-5	4/6/21	34	8	El. 26
B-6	4/6/21	35	8	El. 27
TP-2	4/8/21	33	8	El. 25
Apartment Building.				
B-7	4/6/21	35	8	El. 27
B-8	4/14/21	35	8	El. 27
B-9	4/6/21	36	8	El. 28
Public Storage Building 2				
B-12	4/9/21	33	8	El. 25

Test Boring / Test Pit	Date Recorded	Approx. Surface Elevation (ft)	Depth to Water (ft)	Approx. Water Elevation (feet)
B-13	4/13/21	33	8	El. 25
B-14	4/9/21	33	7	El. 26
B-15	4/13/21	33	8	El. 25
B-16	4/8/21	34	7	El. 27
B-17	4/8/21	34	7	El. 27
B-18	4/7/21	33	8	El. 25
TP-1	4/8/21	33	TP to 5 feet only water not encountered	na

A review of the above table indicates that the water levels vary by about 3 feet in from El. 25 ft. to El. 28 ft across the site. As determined from our subsurface data the water table generally slopes downward to the north towards the bay, which would be expected from the encountered subsurface conditions and area geology.

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 onsite soils underlying the site. 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. 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 Test Borings to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 1140. The results of the Passing No. 200 Sieve tests are presented in Test Boring Logs in Appendix A.

Atterberg Limits

The Atterberg Limits (liquid limit, plastic limit and plasticity index) were determined for representative samples of the site soils at various depths in accordance with Test Method ASTM D 4318 for determination of soil classification and properties. The results of the Atterberg Limit tests are included on the Test Boring Logs enclosed in Appendix A.

Consolidation

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

Loss on Ignition

A Loss on Ignition test was performed on representative samples of the site soils in accordance with Test Method ASTM D 2974 to estimate the organic content of the soil. The results of these tests are presented in the Test Borings, Appendix A.

Unconfined Compressive Strength

The Unconfined Compressive Strength was determined for representative samples of the on-site soil in accordance with Test Method ASTM D 2166. This test method provides an appropriate value of the undrained shear strength of cohesive soils in terms of total stresses. The results of these tests are presented in Test Borings, 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 tested to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water soluble sulfate which could result in chemical attack of cement. The following table presents the results of our laboratory testing.

PARAMETER	**Bulk Sample 1 to 5 ft.**
pH	8.7
Chloride	57 ppm
Sulfate	0.0024%
Resistivity	8,500 ohm-cm

The chloride content of near-surface soils was determined for a composite sample with results of this test indicating that tested soils have a **low exposure to chloride**.

The results of the soil pH test, indicated the tested composite soil sample is slightly basic. The laboratory resistivity test resulted in the tested soils to be **low corrosive potential** when in contact with ferrous materials. 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.

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample. Our laboratory test data indicated that the tested near surface soils contain approximately 0.0024 percent of water soluble sulfates. A negligible exposure to sulfate can be expected for concrete placed in contact with the on-site soils. **Special sulfate resistant cement is not considered necessary** for concrete which will be in contact with the tested on-site soils.

5.0 CONCLUSIONS AND RECOMMENDATIONS

We understand that a 7-story apartment building and two, 5-story Public Storage buildings with no subterranean levels are planned for construction. Based on our subsurface exploration, the site is underlain by soils with compressible clay deposits with some organic content, with layers of gravels and sands.

Conditions imposed by the proposed development have been evaluated on the basis of the structural data presented in Section 2.2 and engineering characteristics of the subsurface materials encountered during our subsurface investigation and their anticipated behavior both during and after construction. Conclusions and recommendations presented for the design of building foundations, building floor slab, and pavements, along with site preparation recommendations and construction considerations are discussed in the following sections of this report.

From a soils engineering perspective, 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.

5.1 Geologic and Seismic Hazards

Active Fault Zones

The site is not located within an a published Alquist-Priolo Earthquake Fault Zone report. 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.

Seismic Hazard Zones

According to the Seismic Hazard Zone report for the Mountain View Quadrangle published by the CGS, the site is located within a liquefaction hazard zone. Additionally, as noted within the Seismic Hazard Zone Report, the historic high groundwater is about 5 feet below grade. Therefore, liquefaction analysis is deemed necessary for this site.

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

5.2 Seismic Design Considerations

Faulting/Seismic Design Parameters

The site is not located within a published Alquist-Priolo Earthquake Fault Zone area. 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. In accordance with *ASCE 7*, Chapter 20, a Site Classification D is recommended for this site based upon the mapped geological features of the site also verified by test borings.

According to the maps of known active fault near-source zones to be used with the 2019 California Building Code (CBC), the Monte Vista-Shannon Fault and North San Andreas faults are the closest known active faults and are located approximately 5.1 and 7.7 miles from the site, respectively. Based upon a deaggregation analysis the Maximum Magnitude (Mw) earthquake is 7.2.

The proposed structure should be designed in accordance with the current versions of the: *California Building Code (CBC)*, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures ASCE 7*, and applicable local codes. The following values are determined by using the web-based SEAOC/OSHPD Seismic Design Map Tool based upon the *CBC 2019* and *ASCE 7-16*:

CBC 2019, Earthquake Loads	
Site Class Definition (Section 1613.2.2)	D
MCER, S_S ,Determined in Section 1613.2.1 for 0.2 second)	1.368
MCER, S_1 Determined in Section 1613.2.1 for 1.0 second)	0.457
Site Coefficient, F_a (Table 1613.2.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613..2.3 (2) 1-second period)	1.843
Site Modified Spectral Acceleration Value, S_{MS} (Eq. 16-36)	1.368
Site Modified Spectral Acceleration Value, S_{M1} (Eq 16-37)	0.842
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-38)	0.912
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-39)	0.561

According to Section 11.4.7 of ASCE 7-16 for structural engineering considerations, a ground motion hazard analysis is required and should be performed in accordance with Section 21.2 for structures on Site Class D with S_1 greater than or equal to 0.2. However, as an exception to performing the ground motion hazard analysis, the value of the Seismic Response Coefficient

(Cs) must be determined by Equation (12.8-2) for values of the fundamental period of the building (T) ≤ 1.5Ts, and taken as 1.5 times the value computed in accordance with either Equation (12.8-3) for $T_L \geq 1.5Ts$, or Equation (12.8-4) for $T > T_L$.

Liquefaction

Our review of the Seismic Hazard Zone Report for the Mountain View Quadrangle indicates that the site is located within a liquefaction hazard zone. In addition, the depth to historic high groundwater is reported to be approximately 5 feet below grade. Accordingly, a detailed liquefaction analysis was deemed appropriate and was performed.

The liquefaction analysis was performed utilizing the computer software program LiquefyPro and based on the 2019 CBC, and California Geological Survey (CGS) Special Publication 117A. For this analysis we used the soil profile identified within boring B-5. The site accelerations (MCE corresponding PGA_M) of 0.657g as obtained from the SEAOC/OSHPD Seismic Design Map Tool and determined from ASCE 7-16. A corresponding site moment magnitude of 7.2 was determined using deaggregation methods published by USGS. Input parameters for blow count data were corrected for borehole diameter, sampling type, automatic hammer type, and depth.

The on-site fine grained soils were evaluated to determine susceptibility to liquefaction during ground shaking in accordance with the criteria outlined within SP117A. 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 very loose to medium dense granular soils.

The soils obtained during our subsurface exploration were tested per SP117A guidelines. Our laboratory results were analyzed to determine potentially liquefiable and non-liquefiable strata to be used in our liquefaction settlement analysis. The following table contains results for preliminary screening of the fine-grained soil layers:

Test Boring No. & Depth	Liquid Limit (LL)	Plastic Index (PI)	In-situ Moisture	W_c/LL
B-5 @ 5 ft. [1]	30	11	21	0.70
B-5 @ 15 ft. [1]	25	7	15	0.60
B-5 @ 20 ft. *	58	39	30	0.52
B-5 @ 25 ft. *	58	39	28	0.48
B-5 @ 30 ft. *	58	39	34	0.59
B-5 @ 40 ft. [1]	26	9	18	0.69
B-5 @ 45 ft. [1]	27	9	8	0.30

*Soil tested and considered to be non-sensitive with Sensitivity ranging from 1.1 to 1.9.

[1] Non-liquefiable.

[2] Potentially liquefiable

The results of our analysis performed at boring B-5 are presented graphically as Plates A-1 of Appendix A. The computer output files are also included. Due to the presence of a thick layer of non-liquefiable soil overlaying the potentially liquefiable layered soils, the most likely impact of soil liquefaction will be ground surface settlement resulting from volumetric strain within the liquefied soil layers. Based on the results of the liquefaction analysis we estimate that ground settlement resulting from the design level earthquake is as follows:

Seismically Induced Settlement (inches)

	B-5 (2% in 50 year)
Total	1.0
Differential	0.5

The magnitude of the calculated liquefaction settlement will decrease significantly with ground improvement methods.

Liquefaction-Induced Lateral Spreading

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

Liquefaction–Induced Potential for Surface Manifestation

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

5.3 Foundation Recommendations

Compressible clay with variable organic contents from less than 4 percent to about 25 percent were encountered within portions of the proposed building areas at variable depths and thickness. The soil is expected to consolidate and settle under new building loads and/or new fill weight if grades are raised, and if a conventional foundation system is constructed without ground improvements. The estimated static differential settlement of at least 2 inches is considered too large for a conventional spread footing foundation or mat/slab design without a ground improvement system.

Several site improvement techniques were explored including removal and replacement, chemically treated soil, select aggregate base fill replacement, reinforced earth, and combinations of these methods of site improvements, however differential settlements in the best case could only be reduced to 2 inches. Therefore, due to the presence of compressible clay, we anticipate that a ground modification program for each building area, with or without grade beams, and a floor slab is being considered.

Bedrock in the area is estimated from geological maps to be several hundred feet deep.

Ground Improvement Systems

A ground improvement program (such as Grouted Aggregate Piers/GeoPiers, VibroPiers, Rigid Inclusions, or similar) is recommended which would then allow for a more conventional shallow foundation or mat/slab design if desirable.

If vibratory installation methods are used, existing structural features and buildings near the site should be surveyed and monitored for pre-existing damages.

For the storage buildings at the site it is expected that the ground improvement spacing of about 10 feet by 10 feet will be under the first floor columns also spaced at 10 feet by 10 feet; for the apartment building it may be necessary to have a more variable layout for the ground improvements in areas of more concentrated column and wall loading.

If the ground improvement option is desired, we recommend that a specialty contractor be consulted to determine the most suitable options. The ground improvement program design should also be provided by the specialty contractor. Once a design has been established by the specialty contractor, we recommend that the design be provided to Giles for review and comment. If a ground improvement program is used, post-improvement test borings are recommended to confirm that the treated soil has been sufficiently improved to allow for the use of a shallow foundation design. If those borings indicate that the soil has not been improved as needed, additional ground improvement efforts will be required.

Following the proper completion of a ground improvement program, we would expect that the building foundations could be designed for a conventional shallow foundation system incorporating an allowable soil bearing pressure of about 3,000 to 5,000 psf, and a modulus of subgrade reaction Ks of 80 to 100 psf/sf, but should be based upon the design of the local ground improvement specialist firms. The specific design of the allowable soil bearing pressure and subgrade modulus will be dependent upon the specifics of the selected ground improvement program and post-improvement testing. The foundation design i.e bearing pressures, modulus, etc. as a result of the ground improvement system will be provided by the ground improvement specialist.

5.4 Floor Slab Recommendations

The slab is anticipated to span between ground improvements and/or grade beams. It is possible that the existing on-site gravel, the crushed aggregate working mat, the aggregate break layer above the ground improvement system, may also be used as the aggregate layer under the structural slab. The design of the floor slab is recommended to be performed by the project structural engineer to ensure proper reinforcing and thickness.

For the ground improvement system, the grade beam and slab-on-grade, or structural mat/slab, should be designed by the structural engineer, based upon the improved allowable soil bearing pressures and parameters provided by the ground improvement specialist.

The structural slab is recommended to be underlain by a minimum 4-inch thick layer of crushed aggregate. Based upon the crushed aggregate beneath the slab a soil coefficient of friction of 0.55 is recommended.

A 15-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.) and control moisture through the floor slab. It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture. The vapor retarder is recommended to be in accordance with ASTM E 1745-97, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Structural Slab Settlement

The post-construction total and differential settlement should be estimated by the ground improvement specialist based on improved soil conditions.

5.5 Construction Considerations

Construction Dewatering

Groundwater was measured at depths ranging from about of about 7 to 8 feet. We anticipate that groundwater or perched water conditions will be encountered during construction. Dependent upon the depth of the excavations, filtered sump pumps placed in pits in the bottoms of excavations at low points are expected to be suitable provided the excavations do not extend more than a few feet below the groundwater. If excavations extending several feet into groundwater are necessary, a more elaborate dewatering system, such as well points may be necessary to facilitate construction.

Some excavations including elevator pits are anticipated to be near the water table which may require subgrade rock stabilization at the bottom of the excavations, and/or waterproofing in these areas.

Soil Excavation

All excavations should be performed in accordance with OSHA guidelines and all applicable local codes, which is the responsibility of the contractor. Localized stability problems may be encountered within vertical excavations due to granular soils. Some water seepage should also be expected.

5.6 Retaining Wall Recommendations

Due to the existing site grades and planned building layout, it is anticipated that minor retaining walls, and below grade structures (elevator pits) will be required.

The retaining wall(s) may be designed as conventional reinforced concrete cantilevered walls supported by spread footings designed for an allowable soil bearing pressures of 2,000 psf and 3,000 psf for footings bearing in the existing fill and/or possible, or as determined by the ground improvement specialist.

Static and Seismic Lateral Earth Pressures

For active, at-rest, and passive conditions we recommend EFP of 50, 75, and 275 pcf, respectively. For retaining structures with retained soil 6 feet in height or great, we recommend that a seismic increment of 20 pcf be added to the static lateral earth pressures.

The above pressures also consider level backfill extending at least 150% of the wall height behind the wall and surface drainage directed away from the wall. Backfill behind the wall should consist of free-draining granular materials. The EFP may be used for on-site soils to be used as backfill materials, assuming drained conditions and a level adjacent backfill extending at least 150% the height of the wall and surface water is directed away from the wall. However, imported soils of low expansion (EI < 51) and/or backfill being placed behind the retaining wall should be tested by the geotechnical engineer prior to placement to verify strength parameters. All retaining walls should be designed with a proper subdrain systems. All walls should also be designed to support any adjacent structural surcharge loads imposed vehicle or structural loading, in addition to the above recommended active earth pressure.

Undrained conditions may occur in below grade retaining structures and therefore these structures must be designed for undrained earth pressures. For the undrained active and at-rest pressures, EFP of 75 and 80 pcf is recommended.

Drainage and Damp-proofing

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

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

Wall Backfill

Retaining wall backfill behind the drainage layers should consist of low-expansive on-site or imported soils (EI < 51), as determined by ASTM D 4829 method, and approved by the geotechnical engineer. Wall backfill should not contain significant organic material, rubble, debris, and rocks or cemented fragments larger than 3 inches in greatest dimension. A 1-foot thick, low-expansive cohesive layer or pavement should be placed at the surface to help prevent surface water intrusion. A geotextile or filter fabric should be placed between the granular drainage layers and adjacent soils (excavated face or compacted materials) to prevent fines from migrating into the drainage layers.

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

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

5.7 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered during our subsurface exploration.

Site Clearing & Demolition

Clearing and demolition operations for the proposed development will include demolition and removal of the existing buildings, pavements, miscellaneous structures, and utilities. Demolition should include removal of all foundations, floor slabs and any below-grade construction.

Clearing should also include the removal of any vegetation and debris within the proposed site development area. Trees, large shrubs, and/or their root systems to be removed should be grubbed out. Abandoned utilities encountered during excavations and grading should also be removed and capped off as described later in this report.

Existing pavement should be removed or processed to a maximum 3-inch size and stockpiled for use as compacted fill or a stabilizing material for the new development. Processed asphalt may be used as fill, sub-base course material, or subgrade stabilization material beyond the building perimeters. Processed concrete may be used as fill, sub-base course material, or subgrade stabilization material both within and outside of the building perimeters. The existing pavement is recommended to remain in-place as long as possible to help protect the subgrade from construction traffic and weather-related disturbance. All soils disturbed by the demolition of the existing improvements should be removed to a suitable subgrade, as determined by the project geotechnical engineer.

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

Existing and New Utilities

All existing utilities should be located. Utilities that will be preserved are recommended to be relocated outside the building areas. Utilities that are not reused should be capped off and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations made for removed utilities 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, grading operations must be carefully performed so as not to disturb or damage the existing utility.

Building Overexcavation

Based on the proposed finish grade for the building relevant to existing grades, grading within the proposed building area to obtain finished subgrade elevations will include cuts and/or fills of up to about 2 feet within the building areas. Based upon the ground improvement methods recommended, overexcavation is not required.

Grading and excavations if required should not be performed within at least a 2:1 (H:V) projection from the ground extents of any Caltrans structural or approach feature along the northern property line. If any excavations are required in these immediate areas, Giles should be contacted to review the planned grading.

Proofroll and Compact

Following lowering of site grades as required to establish finish subgrade elevations, the exposed subgrades in the areas of the buildings, new pavements, and sidewalks should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade, or stabilized. However, proofrolling may be eliminated beneath the buildings where a ground modification system is to be used for conventional foundation or mat/slab, and foundation support. Following proofrolling and completion of any necessary over-excavation, the subgrades should be scarified to a minimum depth of 6 inches, watered or air dried to slightly above optimum moisture content (per ASTM D1557) and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557) maximum density, and 95 percent of the Modified Proctor in the upper 12 inches of pavement subgrade. Low areas and excavations may then be backfilled in lifts with suitable low expansive (EI < 51) structural compacted fill. The selection, placement, and compaction of structural fill should be performed in accordance with the project specifications. The Guide Specifications included in Appendix D (Modified Proctor) of this report are recommended to be used, at a minimum, as an aid in developing the project specifications.

Some type of subgrade improvement may be necessary in select areas since the site was formerly developed, and due to shallow and possible perched groundwater, in areas where the subgrade is subjected to construction traffic disturbance, and if construction is during adverse weather conditions. Subgrade improvement methods might include the use of a crushed-stone “bridging” mat placed on a geotextile or geogrid. It is recommended that specific subgrade improvement recommendations be provided by a geotechnical engineer during construction.

The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, as described in Item No. 5 of the *Guide Specifications*. The optimum moisture content is to be determined by the Modified Proctor compaction test.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced with new fill or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a geotechnical engineer confirms that the previous lift was properly placed and compacted. Subgrade soil will likely need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Reuse of On-site Soil

On-site low expansive soils (EI < 51) material may be reused as structural compacted fill, during favorable weather conditions, within the proposed building and pavement areas provided they do not contain oversized materials (greater than 3 inches) and/or significant quantities of organic matter or other deleterious materials. As an alternative, select import fill (EI < 51) may be used. If desired, it may also be possible to chemically treat the on-site soils to improve their workability during wet weather conditions.

Due to the moist and very moist soils encountered some drying of soil to achieve the required compaction may be necessary.

Import Structural Fill

Select import soils should consist of low expansive (EI < 51) soils with not more than 15 percent passing the No. 200 sieve (silt and clay size). Material designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to plasticity criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure and pavement support characteristics, as well as low soluble sulfate content and corrosivity.

Subgrade Protection

Some of the near surface soils that are expected to comprise the subgrade are moisture and disturbance sensitive. Unstable soil conditions will develop if the soils are exposed to moisture increases or are disturbed (rutted) by construction traffic. The site should be graded to prevent water from ponding within construction areas and/or 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. During wet weather conditions, consideration should be given to limiting construction traffic to specific aggregate “haul routes” to limit wide-spread disturbance of the subgrade.

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

Fill Placement

All structural fill should be placed in 8-inch-thick maximum loose lifts; moisture conditioned and then compacted in place to at least 90 percent (95% for upper 12 inches of pavement subgrade and within the foundation influence zone) of the Modified Proctor (ASTM D1557) maximum density in accordance with the project specifications. A representative of the 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.

Wet and Dry Weather Conditions - Grading

Site preparation and grading activities conducted during dry, fair weather conditions, are not expected to require significant additional over-excavation, due to unstable soil conditions, provided the subgrade is initially in a stable condition with the construction of a working mat as described above. Alternatively, subgrade stability may be achieved by chemical modification of the soils through the addition of fly ash, hydrated lime or Portland cement (depending upon soil type and testing soils sensitivity to modification) followed by proper compaction specialized subgrade stabilization techniques are required, the actual stabilization method should be determined by a representative of the project geotechnical engineer to provide the appropriate recommendations based on field evaluation and testing.

5.8 Pavement Recommendations

Design parameters for any planned paved parking lot areas within the subject property are presented for asphaltic concrete or concrete pavement constructed for driveways, aprons, or in non-structural slab areas.

Subgrades for New Pavement

Following completion of the recommended subgrade preparation procedures, the pavement subgrade soils are expected to consist of silt and clay with variable amounts of fine sand. The anticipated subgrade soils are classified as poor subgrade materials with estimated R Value of about6 10 when properly prepared based on the Unified Soil Classification System designation of CL-CH. An estimated R value of 10 has been used in the preparation of the pavement design based on these soils. It is possible that some on-site excavated or existing soil may be used as a portion of the aggregate base layer.

It should, however, be recognized that the local agency may require a specific R value test to verify the use of the following design. It is recommended that this testing 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 represents the recommended thicknesses for new asphaltic concrete pavement with the appropriate state highway specifications so that the proper materials and construction procedures are used. However, local codes may require specific testing to determine the soil support characteristics and/or minimum pavement section thicknesses. A parking stall pavement section has also been presented. However, if truck traffic cannot be excluded from the parking stalls, the drive pavement section should be used or a reduced service life (premature failure) may occur.

MINIMUM RECOMMENDED PAVEMENT SECTION THICKNESS			
Materials	Asphalt Pavement Section Thickness (Inches)		Washington DOT standard Specifications
	Parking Lot Stalls	Parking Lot Drives	
Asphaltic Concrete Surface Course	1	1	Section 5-04
Asphaltic Concrete Binder Course	2	2	Section 5-04
Aggregate Base	6	8	Compacted to 95% of maximum Dry Density per Modified Proctor to minimum 12 inch depth
Subgrade	12	12	Compacted to 95% of maximum Dry Density per Modified Proctor to minimum 12 inch depth

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

Concrete Pavement

Portland cement concrete pavement is recommended for areas of new pavement that will be subjected to channelized traffic, large loads or intense vehicular stresses such as a trash enclosure loading zone and the entrance/exit aprons. In such areas, a 6-inch thick, properly reinforced concrete pavement is recommended. The concrete pavement is recommended to be underlain by a 6-inch thick compacted coarse granular base placed on a properly prepared subgrade. The use of concrete pavement is also recommended within the entrance/exit aprons to the parking lot. Minimum reinforcement within concrete pavements is recommended to consist of heavy welded wire fabric (6 X 6-W2.9 X W2.9 WWF), placed at mid-slab height.

General Considerations

Pavement designs are based on Caltrans design parameters. It is, therefore, recommended that a representative of the geotechnical engineer observes and test subgrade preparation, and that the subgrade be evaluated immediately before pavement construction.

5.9 Recommended Construction Materials Testing Services

This 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, floor slab, retaining walls, and pavement support soil; concrete; asphalt, 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.

5.10 Basis of Report

This report is based on Giles' proposed scope of work, meetings with the client and project team, and correspondence. 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.

6.0 REFERENCE

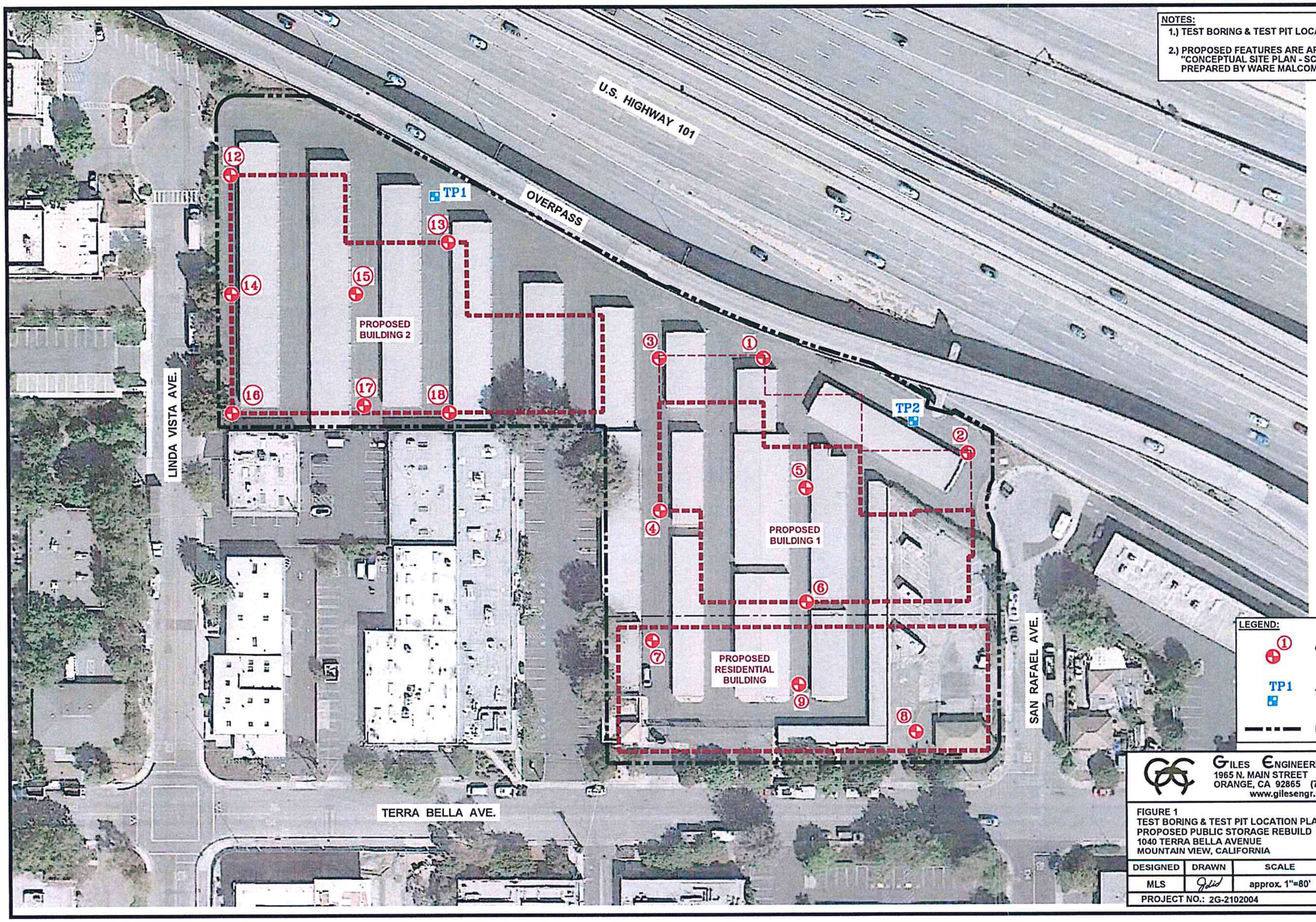
Fisher, Brendan R., Project Engineer, Clark, Michael, CEG1264, Senior Engineer Geologist, Liang, Chalerm (Beeson) S., CE2031, Project Manager, Kleinfelder, Foundation Report, S101 On-Ramp/S101-S85 Separation (Bridge No. 37-0547K), Route 85/101 Interchange Project, Caltrans District 4, Santa Clara County, California, August 12, 2002, File No.: 12-3063-90/SR4

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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



NOTES:

1.) TEST BORING & TEST PIT LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "CONCEPTUAL SITE PLAN - SCHEME 10", DATED 10-12-, 2020, PREPARED BY WARE MALCOMB.

LEGEND:

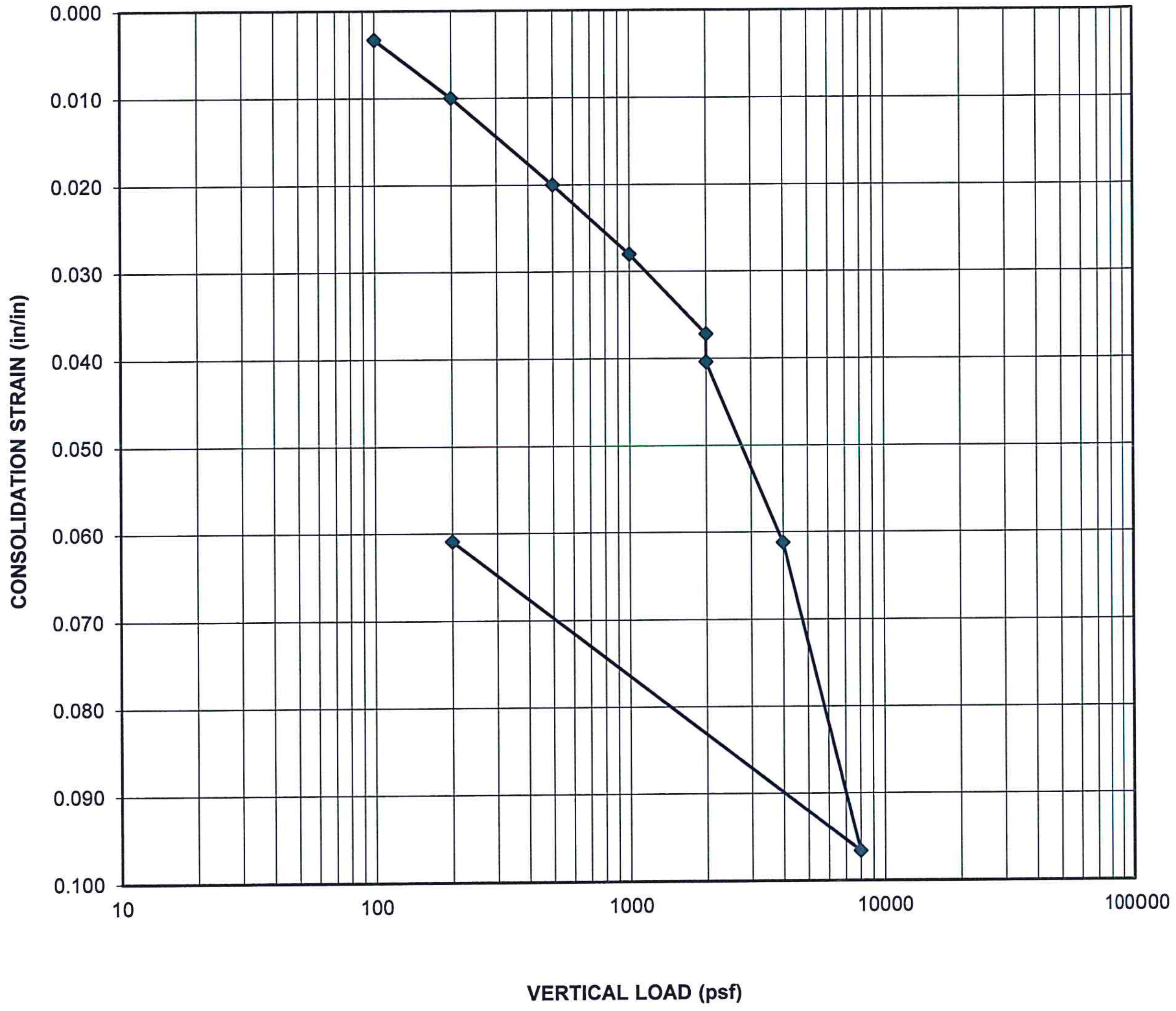
①	GEOTECHNICAL TEST BORING
TP1	TEST PIT LOCATION
—·—·—	PROPERTY LINE

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

FIGURE 1
TEST BORING & TEST PIT LOCATION PLAN
PROPOSED PUBLIC STORAGE REBUILD
1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
MLS	Jdid	approx. 1"=80'	04-20-21	–
PROJECT NO.: 2G-2102004			CAD No. 2g2102004-blp2	

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty Clay
Boring No.	B-1
Sample No.	2-CS
Depth (ft.)	3.5 to 5
Elevation (ft.)	27
Liquid Limit	59
Plastic Limit	21
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	31.8
Final Moisture Content (%)	32.4
Natural Density (pcf)	121.1
Initial Dry Density (pcf)	91.9
Final Dry Density (pcf)	83.0
Hydro-Consolidation at 2,000 psf	0.3%

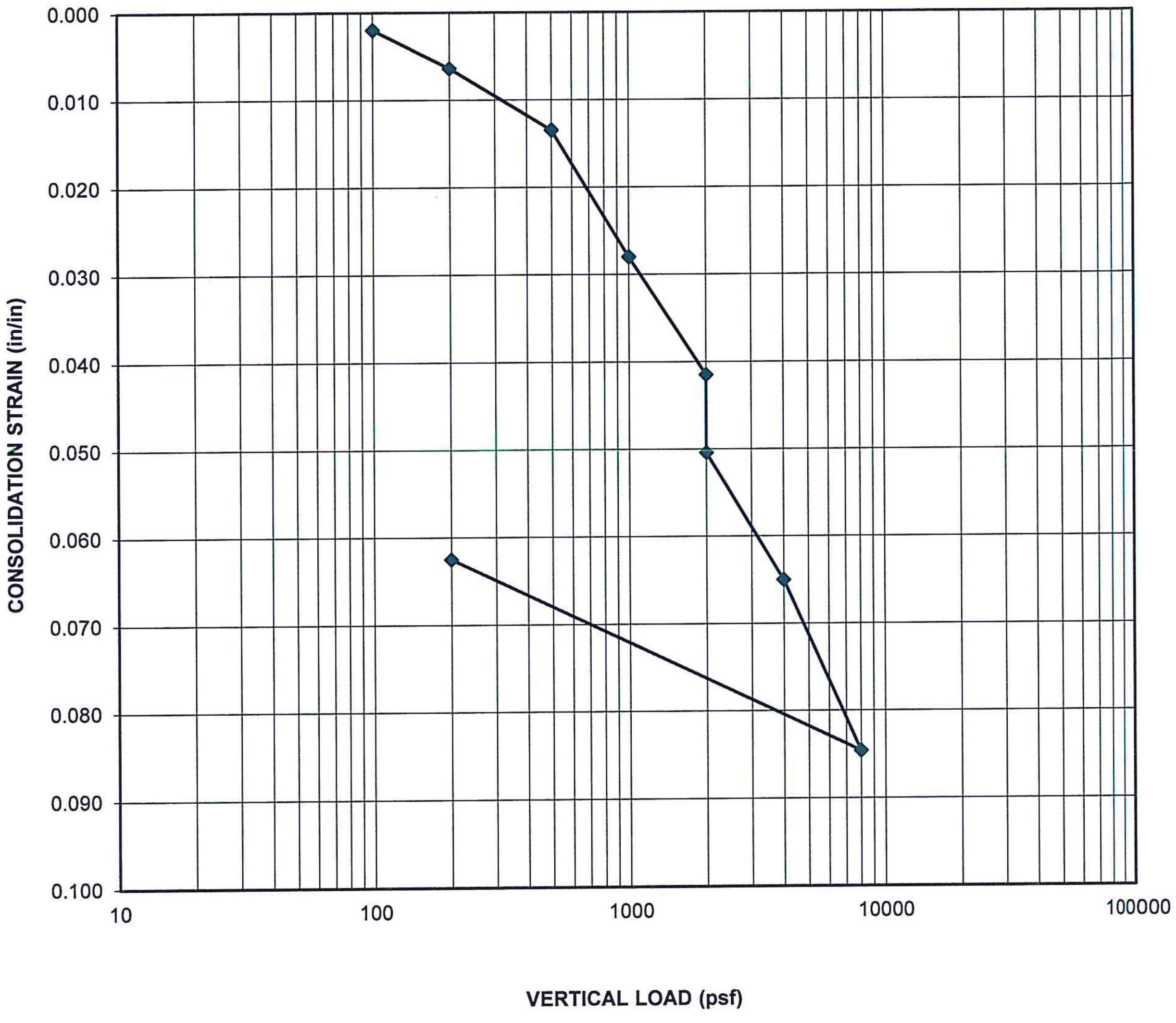
Sample inundated with water at 2,000 psf pressure for observation of swell or hydro-consolidation

Project:	Public Storage Mountain View
Client:	Public Storage
Project No.:	2G-2102004
Figure No.:	B-1 at 3.5 ft

GILES ENGINEERING ASSOCIATES, INC.

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

CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty Clay, trace fine Sand
Boring No.	B-1
Sample No.	3-CS
Depth (ft.)	6 to 7.5
Elevation (ft.)	24.5
Liquid Limit	54
Plastic Limit	19
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	26.7
Final Moisture Content (%)	23.7
Natural Density (pcf)	117.0
Initial Dry Density (pcf)	92.3
Final Dry Density (pcf)	86.5
Hydro-Consolidation at 2,000 psf	0.9%

Sample inundated with water at 2,000 psf pressure for observation of swell or hydro-consolidation

Project:	Public Storage Mountain View
Client:	Public Storage
Project No.:	2G-2102004
Figure No.:	B-1 at 6 ft

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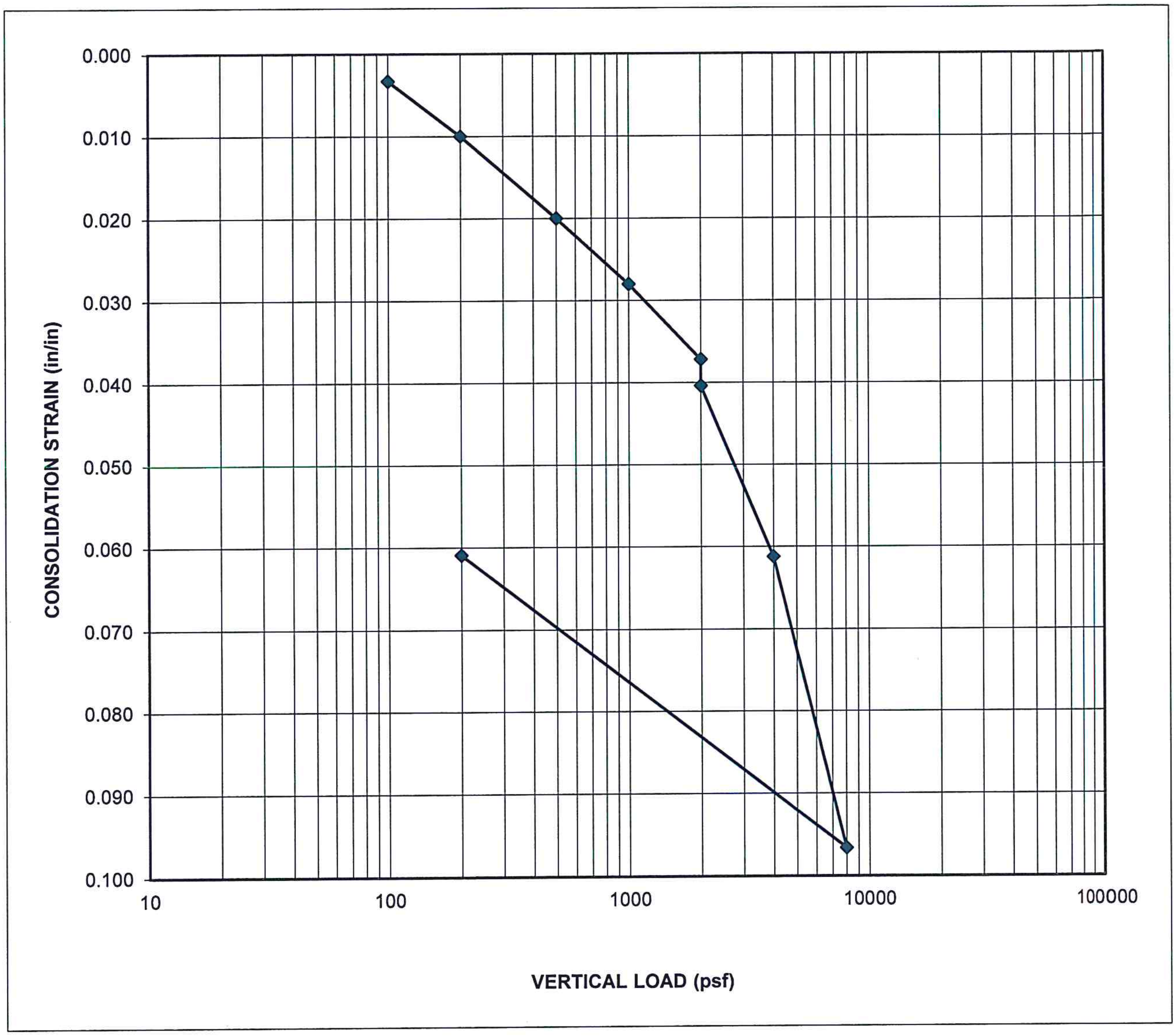
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty Clay
Boring No.	B-1
Sample No.	2-CS
Depth (ft.)	3.5 to 5
Elevation (ft.)	27
Liquid Limit	59
Plastic Limit	21
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	31.8
Final Moisture Content (%)	32.4
Natural Density (pcf)	121.1
Initial Dry Density (pcf)	91.9
Final Dry Density (pcf)	86.3
Hydro-Consolidation at 2,000 psf	0.3%

Sample inundated with water at 2,000 psf pressure for observation of swell or hydro-consolidation

Project:	Public Storage Mountain View
Client:	Public Storage
Project No.:	2G-2102004
Figure No.:	B-1 at 3.5 ft

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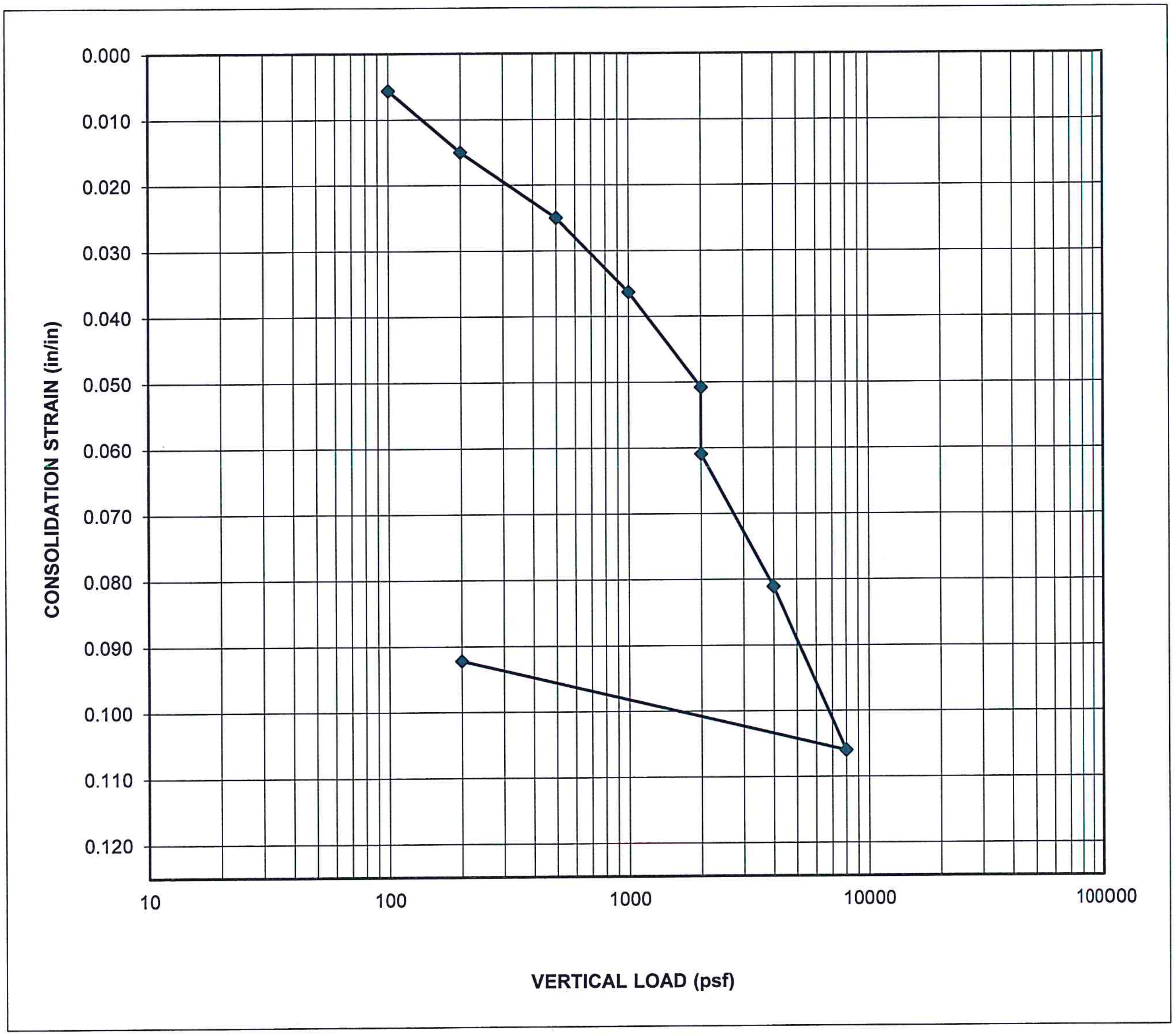
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty Clay
Boring No.	B-6
Sample No.	3-CS
Depth (ft.)	6 to 7.5
Elevation (ft.)	27.5
Liquid Limit	34
Plastic Limit	13
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	23.3
Final Moisture Content (%)	22.5
Natural Density (pcf)	122.5
Initial Dry Density (pcf)	99.3
Final Dry Density (pcf)	90.1
Hydro-Consolidation at 2,000 psf	1.0%

Sample inundated with water at 2,000 psf pressure for observation of swell or hydro-consolidation

Project:	Public Storage Mountain View
Client:	Public Storage
Project No.:	2G-2102004
Figure No.:	B-6 at 6 ft

GILES ENGINEERING ASSOCIATES, INC.

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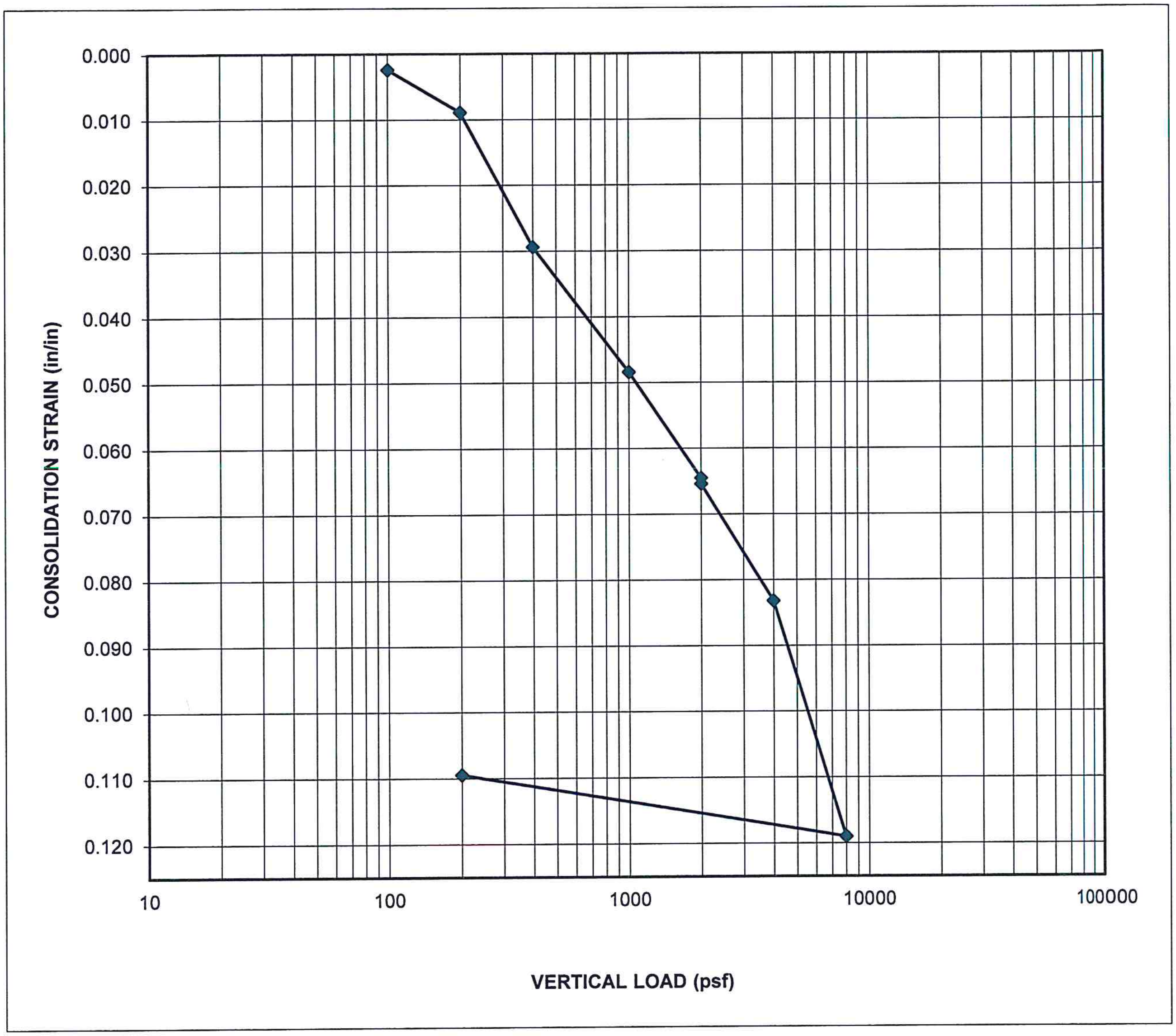
CONSOLIDATION / SWELL / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty Clay trace fine Sand
Boring No.	B-6
Sample No.	4-CS
Depth (ft.)	10 to 11.5
Elevation (ft.)	23.5
Liquid Limit	38
Plastic Limit	20
Specimen Diameter (in.)	2.42
Initial Specimen Thickness (in.)	1.00

Initial Moisture Content (%)	17.6
Final Moisture Content (%)	18.3
Natural Density (pcf)	121.6
Initial Dry Density (pcf)	103.4
Final Dry Density (pcf)	92.0
Hydro-Consolidation at 2,000 psf	0.1%

Sample inundated with water at 2,000 psf pressure for observation of swell or hydro-consolidation

Project:	Public Storage Mountain View
Client:	Public Storage
Project No.:	2G-2102004
Figure No.:	B-6 at 10 ft

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:
B- 1

SURFACE ELEVATION:
32 feet

COMPLETION DATE:
04/06/21

FIELD REP:
J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 5 inches of asphaltic concrete over 6 inches of aggregate base										
Dark Gray Silty Clay - Very Moist (Possible Fill)		30	1-SS	23		4.5+		24		
	5		2-CS	32		3.5		32		Dd=91.9 pcf LL=59 PI=38
Brown to Light Brown Silty Clay, trace fine Sand - Very Moist, sulfur odor	▽	25	3-CS	15		2.0		27		Dd=92.3 pcf LOI=19% LL=54 PI=35
Light Brown Clayey Silt, some Sand (bay mud) - Moist, sulfur odor	10		4-SS	8		2.25		20		LL=70 PI=30 LOI=25%
		20								
Gray Silt with coarse Sand and Gravel - Very Moist	15		5-SS	11				21		LOI=15%
		15								
Brown Sand and Gravel to Brown Silty Sand - Wet	20		6-SS	23				10		
		10								
Gray Silty Clay to Sandy Gravel - Wet	25		7-SS	8				29		
		5								
Gray fine Sand, trace Silt - Wet	30		8-SS	21				15		

Boring Terminated at about 31.5 feet (EL. 0.5')

Water Observation Data

▽	Water Encountered During Drilling: 7 ft.
▽	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

LOI = Loss On Ignition

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

BORING NO. & LOCATION:
B- 2

SURFACE ELEVATION:
33 feet

COMPLETION DATE:
04/06/21

FIELD REP:
J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of asphaltic concrete over 6 inches of aggregate base		30	1-SS	17	4.0	2.75		26		
Dark Brown Silty Clay - Very Moist (Possible Fill)			2-SS	14	3.9	3.75		26		LOI=24%
Brown with White Silty Clay - Very Moist (Possible Fill), sulfur odor	▽		3-SS	9	2.0	2.0		15		
Dark Brown Silty Clay, trace Sand - Very Moist (Possible Fill)	10		4-SS	7		0.75		23		
Greenish Brown fine Sandy Clay - Very Moist		20								
Greenish Brown Clayey Silt, trace coarse Sand - Very Moist			5-SS	12				21		
Blue fine to coarse Sandy Silt, trace Clay and Gravel - Very Moist	20	10	6-SS	5				19		
Brown Gravelly medium to coarse Sand - Wet to Brown Clayey fine Sand - Wet			7-SS	14				15		
Blue Silty Clay - Very Moist	30	0	8-SS	4				28		
Bluish Gray Silty Clay - Wet			9-SS	8				26		
Brown to Blue Silty Sand - Wet	40		10-SS	22				16		

Boring Terminated at about 41.5 feet (EL. -8.5')

Water Observation Data

▽ Water Encountered During Drilling: 7 ft.
▽ Water Level At End of Drilling:
░ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
▒ Cave Depth After Drilling:

Remarks:

SS - Standard Penetration Test

LOI = Loss On Ignition

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

BORING NO. & LOCATION: B- 3

SURFACE ELEVATION: 32 feet

COMPLETION DATE: 04/06/21

FIELD REP: J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of asphaltic concrete over 6 inches of aggregate base										
Dark Brown Silty Clay - Very Moist (Possible Fill)		30	1-SS	18	3.1	3.6		29		
Mottled Brown Silty Clay - Very Moist (Possible Fill)	5		2-SS	15		3.0		30		
Brown Silty Clay - Very Moist	▽	25	3-CS	15		3.0		28		
Light Brown Silty Clay - Very Moist	10	20	4-CS	15		3.5		23		
Brown Silty fine Sand to Clay and Sand - Wet	15	15	5-SS	24				23		
Brown Silty Clay - Very Moist	20	10	6-SS	7				14		
Blue Silty Clay - Very Moist	25	5	7-SS	11				29		
Gray Silty Sand - Moist	30		8-SS	10				18		

Boring Terminated at about 31.5 feet (EL. 0.5')

	Water Observation Data
▽	Water Encountered During Drilling: 7 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

CS = California Split Spoon

SS = Standard Penetration Test

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

BORING NO. & LOCATION: B- 4	
SURFACE ELEVATION: 33 feet	
COMPLETION DATE: 04/06/21	
FIELD REP: J. MAIER/M. KORDAVI	

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphaltic concrete over 4 inches of aggregate base			1-SS	20	1.2	3.0		27		
Brown Silty Clay - Very Moist (Possible Fill)		30								
	5		2-SS	17		3.0		28		
Gray Silty Clay - Very Moist, sulfur odor			3-SS	14	1.7	2.8		28		
	▽	25								
	10		4-SS	6	1.0	1.0		25		LOI=10%
		20								
Brown Sandy Silt, trace Gravel - Very Moist	15		5-SS	11				17		
		15								
Brown Silty and Sandy Gravel - Wet	20		6-SS	20				12		
		10								
Brown Silty Clay to Silty Gravel - Wet	25		7-SS	10				15		LOI=6%
		5								

Boring Terminated at about 29 feet (EL. 4')

	Water Observation Data
▽	Water Encountered During Drilling: 8 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

LOI = Loss On Ignition

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

BORING NO. & LOCATION: B- 5	# TEST BORING LOG
SURFACE ELEVATION: 34 feet	PUBLIC STORAGE FACILITY
COMPLETION DATE: 04/06/21	1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA
FIELD REP: J. MAIER/M. KORDAVI	PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2.5 inches of asphaltic concrete over 3 inches of aggregate base			1-SS	20	4.5	4.5		18		
Gray Silty Clay - Moist (Possible Fill)		30	2-SS	18	4.3	4.0		28		
Brown Silty Clay - Very Moist (Possible Fill)	▽		3-SS	11	2.25	3.0		21		LL=30 PI=11
Gray Clayey Silt - Very Moist	10	20	4-SS	5	0.25			15		LL=25 PI=7
Gray Silty fine Sand - Very Moist			5-SS	17				20		P_{200}=29%
Gray Silty Clay, trace Sand and Gravel - Very Moist	20		6-SS	10				30		
		10	7-SS	9				28		LL=58 PI=39
	30	0	8-SS	12				34		
Brown Silty Sand and Gravel - Wet			9-SS	35				35		P_{200}=28%
Gray Clayey Silt, trace Gravel - Wet	40	-10	10-SS	22				18		LL=26 PI=9
Gray Silty Clay with fine Sand, trace Gravel - Very Moist			11-SS	13				8		LL=27 PI=9
Brown Silty Clay, trace Sand - Very Moist	50		12-SS	16				22		

Boring Terminated at about 51.5 feet (EL. -17.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 8 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- 6	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 35 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/06/21	1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA	
FIELD REP: J. MAIER/M. KORDAVI	PROJECT NO: 2G-2102004	

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 6 inches of aggregate base			1-SS	14	4.2	4.5		27		
Dark Gray Silty Clay - Very Moist (Possible Fill)										
Gray Silty Clay - Very Moist (Possible Fill)	5	30	2-SS	18		2.5		26		
			3-CS	24		4.5		23		Dd=99.3 pcf LL=34 PI=21
	∇									
Brown Silty Clay, trace fine Sand - Wet	10	25	4-CS	15		1.0		18		Dd=103.4 pcf LL=38 PI=18 LOI=11%
Brown Gravelly Sand - Wet	15	20	5-CS	28				13		
Gray Silty Clay - Very Moist	20	15	6-SS	9				29		
Gray Silt to Gravelly Sand - Wet	25	10	7-SS	7				14		
Gray Silty Clay - Very Moist	30	5	8-SS	13				26		LOI=4%

Boring Terminated at about 31.5 feet (EL. 3.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 8 ft.	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
░	Cave Depth At End of Drilling:	LOI = Loss On Ignition
▼	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

SURFACE ELEVATION: 35 feet

COMPLETION DATE: 04/06/21

FIELD REP: J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of asphaltic concrete over 6 inches of aggregate base										
Brown Silty Clay, trace Sand - Moist (Fill)			1-SS	12	1.5	3.5		19		
No recovery	5	30	2-SS	8						
Brown Silty Clay, trace Sand - Moist (Possible Fill)	▽		3-SS	16	3.4	3.5		26		
Brown Silty Clay - Very Moist	10	25	4-SS	6	1.0	1.5		21		
Gray Sandy Silt - Very Moist	15	20	5-SS	12				36		
Gray coarse Sand and Gravel - Wet	20	15	6-SS	29				9		
Gray Silty Clay - Very Moist	25	10	7-SS	6				23		
Gray coarse Sand and Gravel - Wet	30	5	8-SS	11				15		

Boring Terminated at about 31.5 feet (EL. 3.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 8 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- 8

SURFACE ELEVATION:
36 feet

COMPLETION DATE:
04/14/21

FIELD REP:
J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2.5 inches of asphaltic concrete over 4 inches of aggregate base		35	1-SS	15						
Dark Brown Silty Clay - Moist (Possible Fill)										
Brown Silty Clay - Very Moist (Possible Fill)			2-SS	13						
	5	30	3-SS	12						
	▽									
Light Brown Silty Clay - Very Moist	10	25	4-SS	7						
Gray Silty Clay to Sandy Gravel - Wet	15	20	5-SS	28						
Gray Silty Clay - Very Moist	20	15	6-SS	10						
	25	10	7-SS	12						
	30	5	8-SS	12						

Boring Terminated at about 31.5 feet (EL. 4.5')

	Water Observation Data
▽	Water Encountered During Drilling: 8 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

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: 36 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/14/21	1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA	
FIELD REP: J. MAIER/M. KORDAVI	PROJECT NO: 2G-2102004	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 2.5 inches of asphaltic concrete over 4 inches of aggregate base			1-SS	16	2.9	4.5		29		
Gray Silty Clay - Moist (Possible Fill)			2-SS	15	2.0	2.5		29		
Light Brown Silty Clay - Moist (Possible Fill)		30	3-SS	11	2.7	2.5		25		
Brown Silty Clay - Moist (Possible Fill)	∇									
Light Brown Silty Clay - Very Moist	10		4-SS	7		1.6		27		
Gray Silty Sand to Silty Gravel - Very Moist		20	5-SS	37				25		
Gray Silty Clay - Very Moist	20		6-SS	10				27		
Gray Silty Clay, trace Sand - Very Moist		10	7-SS	13				28		
	30		8-SS	12				30		
Gray Silty Clay - Very Moist		0	9-SS	10				31		LOI=8%
Gray Silty Clay, trace Sand - Moist	40		10-SS	11				28		
Gray Silty Gravel - Wet		-10	11-SS	17				11		
Gray Silty Clay, trace Gravel - Very Moist	50		12-SS	11				30		

Boring Terminated at about 51.5 feet (EL. -15.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 8 ft.	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	LOI = Loss On Ignition
░	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-12

SURFACE ELEVATION: 33 feet

COMPLETION DATE: 04/09/21

FIELD REP: J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3.5 inches of asphaltic concrete over 6 inches of aggregate base			1-SS	25	4.2	4.8		29		
Brown Silty Clay, trace Sand - Moist (Possible Fill)		30	2-SS	16		2.5		42		
Light Brown Silty Clay - Moist (Possible Fill)	▽		3-SS	16	3.9	2.8		32		
Brown Silty Clay - Moist										
Brown Silty Clay, trace Sand - Very Moist	10	20	4-CS	18		1.25		27		Dd=97.6 pcf LL=35 PI=19
Gray Silty Clay - Very Moist			5-CS	15				28		
	20	10	6-SS	10				24		
Gray Silty Clay, trace Gravel - Very Moist			7-SS	9				16		
Gray Silty Clay with Gravel - Very Moist	30	0	8-SS	16				18		
Gray Silty Clay - Very Moist			9-SS	9				27		
	40		10-SS	16				24		

Boring Terminated at about 41.5 feet (EL. -8.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 8 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-13	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 33 feet	PUBLIC STORAGE FACILITY	
COMPLETION DATE: 04/13/21	1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA	
FIELD REP: J. MAIER/M. KORDAVI	PROJECT NO: 2G-2102004	

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 4 inches of aggregate base	■										
Gray Silty Clay - Very Moist (Possible Fill)	▨		30	1-SS	15	3.5	3.0		24		
Brown Silty Clay - Moist (Possible Fill)	▨			2-SS	17		4.5		30		
Gray Silty Clay - Moist (Possible Fill)	▨	∇		3-SS	11	3.3	3.0		32		
Dark Gray Silty Clay - Very Moist	▨	10	20	4-SS	6	1.2	1.0		19		
Gray Sandy Gravel - Wet	●			5-SS	33				34		
	●	20	10	6-SS	35				10		
	●								22		
Gray Silty Clay to Sandy Gravel - Wet	▨	30	0	8-SS	16				6		
Gray Sandy Silt - Wet	▥			9-SS	19				11		

Boring Terminated at about 36.5 feet (EL. -3.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 8 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-14	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 33 feet	PUBLIC STORAGE FACILITY 1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA PROJECT NO: 2G-2102004	
COMPLETION DATE: 04/09/21		
FIELD REP: J. MAIER/M. KORDAVI		

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
Approximately 3.5 inches of asphaltic concrete over 6 inches of aggregate base			1-SS	23	5.2	4.5		23		
Brown Silty Clay - Moist (Possible Fill)		30	2-SS	25	2.5	4.5		22		
Brown Silty Clay - Moist	▽		3-SS	15	3.1	3.5		25		
Gray Silty Clay - Very Moist	10		4-SS	10		0.75		30		
		20								
			5-SS	8				31		
Gray Silty Clay, trace Sand - Very Moist	20		6-SS	7				23		
		10								
			7-SS	11				12		
Gray Silty Clay - Very Moist	30		8-SS	9				22		
		0								
			9-SS	12				35		

Boring Terminated at about 36.5 feet (EL. -3.5')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 7 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-15

SURFACE ELEVATION:
33 feet

COMPLETION DATE:
04/13/21

FIELD REP:
J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 4.5 inches of aggregate base		30	1-SS	12	3.5	4.5		30		LOI=12%
Brown Silty Clay - Very Moist (Possible Fill)			2-SS	13	3.9	3.5		22		
Brown Silty Clay - Moist	▽		3-SS	10	2.1	2.6		21		
Gray with Dark Brown Silty Clay - Very Moist	10	20	4-SS	11	1.4	1.5		21		
Brown Sandy Gravel - Wet			5-SS	20				11		
	20	10	6-SS	12				7		
Brown Sandy Silt to Sandy Gravel - Wet			7-SS	16				12		
Brown Sandy Gravel - Wet	30	0	8-SS	15				6		
Gray Sandy Clay - Very Moist			9-SS	8				29		LOI=6%
Gray Silty Clay - Very Moist	40	-10	10-SS	12						
Gray Silty Sand - Very Moist			11-SS	16				15		
Gray Silty Clay, trace Sand - Very Moist	50		12-SS	18				18		

Boring Terminated at about 51.5 feet (EL. -18.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 8 ft.	SS = Standard Penetration Test LOI = Loss On Ignition
▼	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-16

SURFACE ELEVATION:
34 feet

COMPLETION DATE:
04/08/21

FIELD REP:
J. MAIER/M. KORDAVI

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION		Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 6 inches of aggregate base Brown Silty Clay - Moist				1-SS	23	5.1	5.0		6		
Brown Silty Clay, trace Sand - Moist		5	30	2-SS	22		4.5		19		
Gray Silty Clay - Very Moist	▽		25	3-SS	15	2.5	2.6		23		
Brown Silty Clay, trace Sand - Very Moist		10	20	4-SS	11		0.75		21		
Brown Silty Clay - Very Moist		15	15	5-SS	11				19		
Gray Silty Clay - Very Moist		20	10	6-SS	12				24		
		25	5	7-SS	19				12		
Brown Sandy Gravel - Wet		30		8-SS	19				18		

Boring Terminated at about 31.5 feet (EL. 2.5')

	Water Observation Data
▽	Water Encountered During Drilling: 7 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

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

BORING NO. & LOCATION: B-17	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 34 feet	PUBLIC STORAGE FACILITY 1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA PROJECT NO: 2G-2102004	
COMPLETION DATE: 04/08/21		
FIELD REP: J. MAIER/M. KORDAVI		

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 6 inches of aggregate base			1-SS	14	4.1	4.0		24		
Brown Silty Clay - Very Moist (Possible Fill)		30	2-SS	12	2.5	3.0		24		
Brown Silty Clay - Moist	▽		3-SS	8	1.8	1.8		23		
	10		4-SS	6		0.5		22		LOI=3%
		20								
Brown Silty Sand, trace Gravel - Wet			5-SS	25				14		
Brown Silty Clay, trace Sand - Very Moist	20		6-SS	4				28		
		10								
Gray coarse Sand and Gravel - Wet			7-SS	25				16		
Brown Silty Clay, trace Sand - Very Moist	30		8-SS	7				19		
		0								
Brown Silty Clay - Very Moist			9-SS					21		
Brown Silty Clay - Wet	40		10-SS	4				24		

Boring Terminated at about 41.5 feet (EL. -7.5')

	Water Observation Data
▽	Water Encountered During Drilling: 7 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

LOI = Loss On Ignition

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

BORING NO. & LOCATION: B-18	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 33 feet	PUBLIC STORAGE FACILITY 1020 & 1040 TERRA BELLA AVENUE MOUNTAIN VIEW, CA PROJECT NO: 2G-2102004	
COMPLETION DATE: 04/07/21		
FIELD REP: J. MAIER/M. KORDAVI		

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 6 inches of aggregate base										
Gray Silty Clay - Moist (Possible Fill)		30	1-SS	14	3.9	3.5		20		
Gray with White Silty Clay - Moist (Possible Fill)	5		2-SS	15	3.9	2.8		17		
Gray Silty Clay - Moist			3-SS	9	2.0	1.8		23		
Light Brown Silty Clay - Wet	∇	25								
Gray Silty Clay, trace Gravel and Sand - Very Moist	10	20	4-SS	7		0.5		18		
Gray Sandy Gravel - Wet	15	15	5-SS	17				13		
Light Brown Sandy Silt - Very Moist	20	10	6-SS	5				16		
No sample recovery	25	5	7-GRAB					10		
	30		8-SS	5				19		LOI=3%

Boring Terminated at about 31.5 feet (EL. 1.5')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 8 ft.	SS = Standard Penetration Test LOI = Loss On Ignition
▼	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:
TP-1

SURFACE ELEVATION:
33 feet

COMPLETION DATE:
04/08/21

FIELD REP:
JOHN MAIER

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 10 inches of aggregate base		32.5								
Dark Brown Silty Clay, trace debris (nails, wood, asphaltic concrete and concrete) - Very Moist (Fill)	2.5	30.0	1-SS							
Light Brown Silty Clay - Very Moist			2-SS							
Boring Terminated at about 5 feet (EL. 28')	5.0									

	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: TP-2

SURFACE ELEVATION: 33 feet

COMPLETION DATE: 04/08/21

FIELD REP: JOHN MAIER

TEST BORING LOG

PUBLIC STORAGE FACILITY

1020 & 1040 TERRA BELLA AVENUE
MOUNTAIN VIEW, CA

PROJECT NO: 2G-2102004

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 12 inches of aggregate base										
Dark Gray Silty Clay, trace debris (glass) - Moist (Fill)			1-SS							
Blue to Brown fine Sandy Clay - Very Moist (Fill)		30	2-SS							
Light Brown Silty Clay - Moist (Native)			3-SS							
Blue to Brown with White Silty Clay, trace Sand - Very Moist	5		4-SS							
Light Brown Silty Clay, trace fine Sand - Wet	▽	25								

Boring Terminated at about 8.5 feet (EL. 24.5')

	Water Observation Data
▽	Water Encountered During Drilling: 8 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

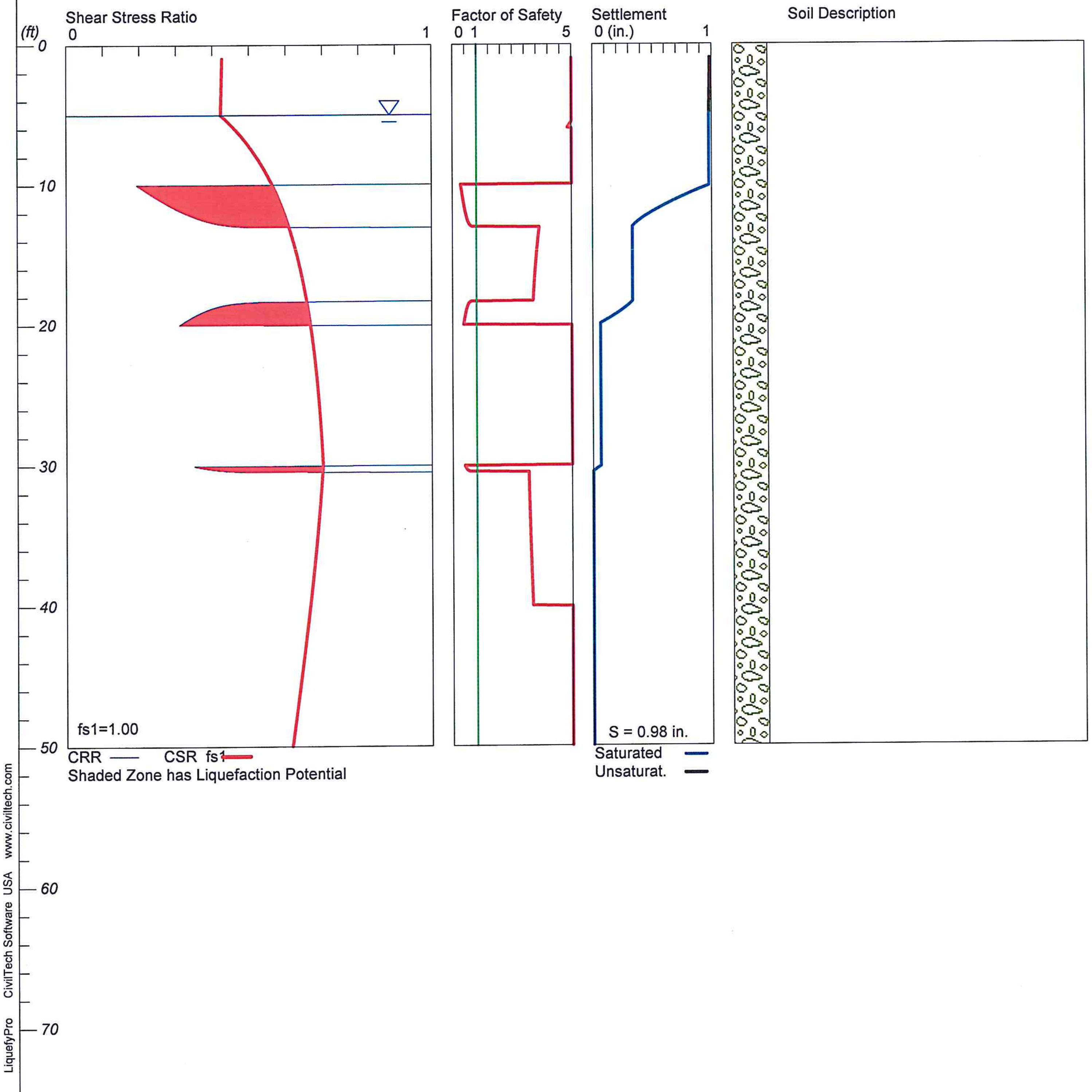
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

Mountain View

Hole No.=B-5   Water Depth=5 ft

Magnitude=7.2
Acceleration=0.657g



```
                              B-5 Liq 2 in 50 yr

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

                                         Copyright by CivilTech Software
                                            www.civiltechsoftware.com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        Font: Courier New, Regular, Size 8 is recommended for this report.
        Licensed to ,   9/14/2021       1:16:22 PM

        Input File Name: P:\John Maier\1 - Public Storage\Geo\Mountain
View\Liq\B-5 Liq 2 in 50 yr.liq
        Title:  Mountain View
        Subtitle:  1020-1040 Terra Bella Ave

        Surface Elev.=
        Hole No.=B-5
        Depth of Hole= 50.00 ft
        Water Table during Earthquake= 5.00 ft
        Water Table during In-Situ Testing= 7.00 ft
        Max. Acceleration= 0.66 g
        Earthquake Magnitude= 7.20

 Input Data:
        Surface Elev.=
        Hole No.=B-5
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 5.00 ft
        Water Table during In-Situ Testing= 7.00 ft
        Max. Acceleration=0.66 g
        Earthquake Magnitude=7.20
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

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

        In-Situ Test Data:
```

```
                        B-5 Liq 2 in 50 yr
        Depth   SPT     gamma   Fines
        ft              pcf     %
        ____________________________________
        1.00    20.00   120.00  90.00
        3.50    18.00   120.00  90.00
        6.00    11.00   120.00  NoLiq
        10.00   5.00    120.00  NoLiq
        15.00   17.00   120.00  29.00
        20.00   10.00   120.00  NoLiq
        25.00   9.00    120.00  NoLiq
        30.00   12.00   120.00  NoLiq
        35.00   35.00   120.00  28.00
        40.00   22.00   120.00  NoLiq
        45.00   13.00   120.00  NoLiq
        50.00   16.00   120.00  NoLiq
        ____________________________________

Output Results:
        Settlement of Saturated Sands=0.98 in.
        Settlement of Unsaturated Sands=0.01 in.
        Total Settlement of Saturated and Unsaturated Sands=0.98 in.
        Differential Settlement=0.492 to 0.650 in.

        Depth   CRRm    CSRfs   F.S.    S_sat.  S_dry   S_all
        ft                              in.     in.     in.
        ____________________________________________________
        1.00    2.22    0.43    5.00    0.98    0.01    0.98
        1.50    2.22    0.43    5.00    0.98    0.01    0.98
        2.00    2.22    0.43    5.00    0.98    0.01    0.98
        2.50    2.22    0.42    5.00    0.98    0.01    0.98
        3.00    2.22    0.42    5.00    0.98    0.00    0.98
        3.50    2.22    0.42    5.00    0.98    0.00    0.98
        4.00    2.22    0.42    5.00    0.98    0.00    0.98
        4.50    2.22    0.42    5.00    0.98    0.00    0.98
        5.00    2.22    0.42    5.00    0.98    0.00    0.98
        5.50    2.22    0.44    5.00    0.98    0.00    0.98
        6.00    2.00    0.46    5.00    0.98    0.00    0.98
        6.50    2.00    0.48    5.00    0.98    0.00    0.98
        7.00    2.00    0.49    5.00    0.98    0.00    0.98
        7.50    2.00    0.51    5.00    0.98    0.00    0.98
        8.00    2.00    0.52    5.00    0.98    0.00    0.98
        8.50    2.00    0.53    5.00    0.98    0.00    0.98
        9.00    2.00    0.54    5.00    0.98    0.00    0.98
        9.50    2.00    0.55    5.00    0.98    0.00    0.98
        10.00   0.19    0.56    0.34*   0.98    0.00    0.98
        10.50   0.22    0.57    0.39*   0.86    0.00    0.86
        11.00   0.25    0.58    0.43*   0.73    0.00    0.73
        11.50   0.28    0.59    0.48*   0.61    0.00    0.61
        12.00   0.32    0.60    0.54*   0.50    0.00    0.50
        12.50   0.37    0.60    0.62*   0.41    0.00    0.41
        13.00   0.50    0.61    0.81*   0.34    0.00    0.34
```

```
13.50    2.22     0.61     3.61     0.33     0.00     0.33
14.00    2.22     0.62     3.58     0.33     0.00     0.33
14.50    2.22     0.63     3.55     0.33     0.00     0.33
15.00    2.22     0.63     3.52     0.33     0.00     0.33
15.50    2.22     0.64     3.49     0.33     0.00     0.33
16.00    2.22     0.64     3.47     0.33     0.00     0.33
16.50    2.22     0.64     3.45     0.33     0.00     0.33
17.00    2.22     0.65     3.43     0.33     0.00     0.33
17.50    2.22     0.65     3.41     0.33     0.00     0.33
18.00    2.22     0.66     3.39     0.33     0.00     0.33
18.50    0.45     0.66     0.68*    0.32     0.00     0.32
19.00    0.38     0.66     0.57*    0.25     0.00     0.25
19.50    0.34     0.66     0.51*    0.15     0.00     0.15
20.00    0.31     0.67     0.46*    0.06     0.00     0.06
20.50    2.00     0.67     5.00     0.06     0.00     0.06
21.00    2.00     0.67     5.00     0.06     0.00     0.06
21.50    2.00     0.68     5.00     0.06     0.00     0.06
22.00    2.00     0.68     5.00     0.06     0.00     0.06
22.50    2.00     0.68     5.00     0.06     0.00     0.06
23.00    2.00     0.68     5.00     0.06     0.00     0.06
23.50    2.00     0.68     5.00     0.06     0.00     0.06
24.00    2.00     0.69     5.00     0.06     0.00     0.06
24.50    2.00     0.69     5.00     0.06     0.00     0.06
25.00    2.00     0.69     5.00     0.06     0.00     0.06
25.50    2.00     0.69     5.00     0.06     0.00     0.06
26.00    2.00     0.69     5.00     0.06     0.00     0.06
26.50    2.00     0.69     5.00     0.06     0.00     0.06
27.00    2.00     0.69     5.00     0.06     0.00     0.06
27.50    2.00     0.70     5.00     0.06     0.00     0.06
28.00    2.00     0.70     5.00     0.06     0.00     0.06
28.50    2.00     0.70     5.00     0.06     0.00     0.06
29.00    2.00     0.70     5.00     0.06     0.00     0.06
29.50    2.00     0.70     5.00     0.06     0.00     0.06
30.00    2.00     0.70     5.00     0.06     0.00     0.06
30.50    2.22     0.70     3.17     0.00     0.00     0.00
31.00    2.22     0.70     3.18     0.00     0.00     0.00
31.50    2.22     0.70     3.19     0.00     0.00     0.00
32.00    2.22     0.70     3.19     0.00     0.00     0.00
32.50    2.22     0.69     3.20     0.00     0.00     0.00
33.00    2.22     0.69     3.21     0.00     0.00     0.00
33.50    2.22     0.69     3.22     0.00     0.00     0.00
34.00    2.22     0.69     3.22     0.00     0.00     0.00
34.50    2.22     0.69     3.23     0.00     0.00     0.00
35.00    2.22     0.69     3.24     0.00     0.00     0.00
35.50    2.22     0.68     3.25     0.00     0.00     0.00
36.00    2.22     0.68     3.26     0.00     0.00     0.00
36.50    2.22     0.68     3.27     0.00     0.00     0.00
37.00    2.22     0.68     3.28     0.00     0.00     0.00
37.50    2.22     0.68     3.29     0.00     0.00     0.00
38.00    2.22     0.67     3.30     0.00     0.00     0.00
38.50    2.22     0.67     3.31     0.00     0.00     0.00
```

```
        39.00   2.22    0.67    3.32    0.00    0.00    0.00
        39.50   2.22    0.67    3.33    0.00    0.00    0.00
        40.00   2.22    0.66    3.34    0.00    0.00    0.00
        40.50   2.00    0.66    5.00    0.00    0.00    0.00
        41.00   2.00    0.66    5.00    0.00    0.00    0.00
        41.50   2.00    0.66    5.00    0.00    0.00    0.00
        42.00   2.00    0.66    5.00    0.00    0.00    0.00
        42.50   2.00    0.65    5.00    0.00    0.00    0.00
        43.00   2.00    0.65    5.00    0.00    0.00    0.00
        43.50   2.00    0.65    5.00    0.00    0.00    0.00
        44.00   2.00    0.65    5.00    0.00    0.00    0.00
        44.50   2.00    0.64    5.00    0.00    0.00    0.00
        45.00   2.00    0.64    5.00    0.00    0.00    0.00
        45.50   2.00    0.64    5.00    0.00    0.00    0.00
        46.00   2.00    0.64    5.00    0.00    0.00    0.00
        46.50   2.00    0.63    5.00    0.00    0.00    0.00
        47.00   2.00    0.63    5.00    0.00    0.00    0.00
        47.50   2.00    0.63    5.00    0.00    0.00    0.00
        48.00   2.00    0.63    5.00    0.00    0.00    0.00
        48.50   2.00    0.62    5.00    0.00    0.00    0.00
        49.00   2.00    0.62    5.00    0.00    0.00    0.00
        49.50   2.00    0.62    5.00    0.00    0.00    0.00
        50.00   2.00    0.62    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.

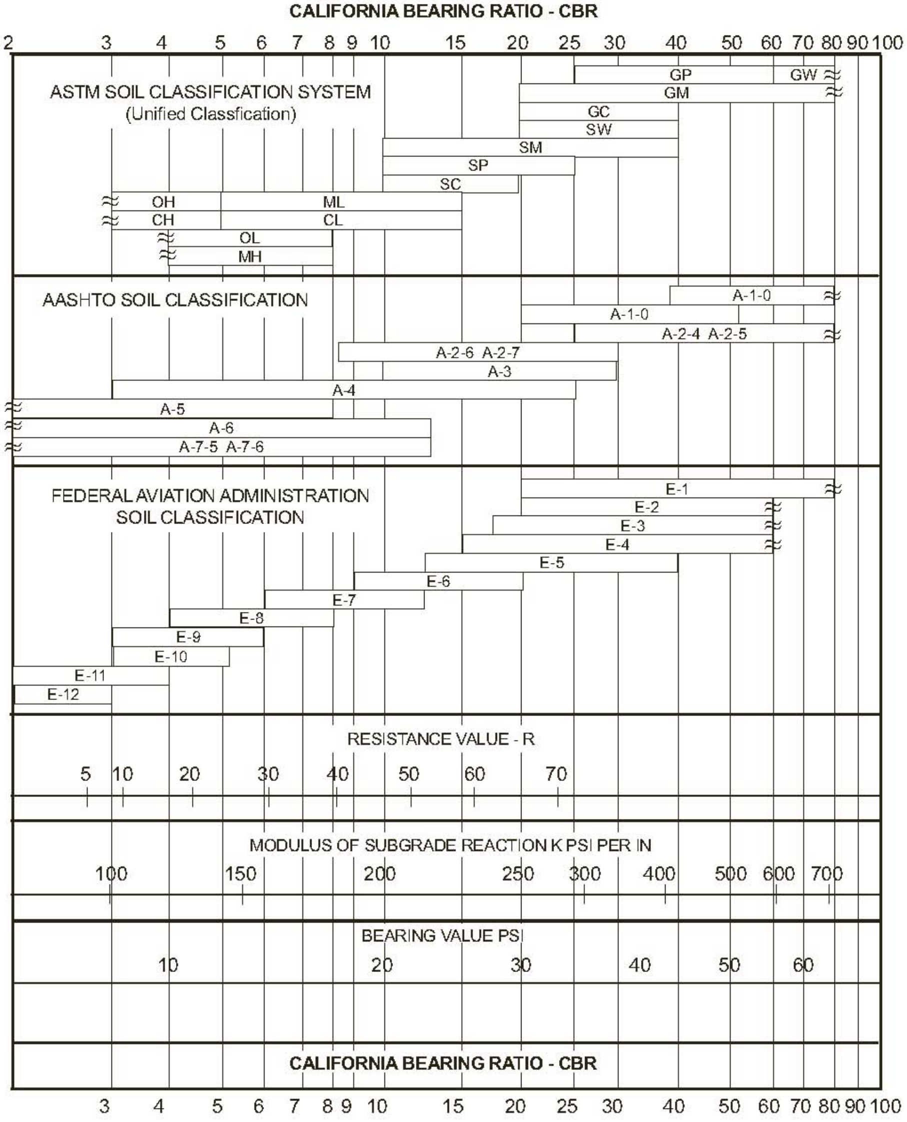
Laboratory Testing

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.

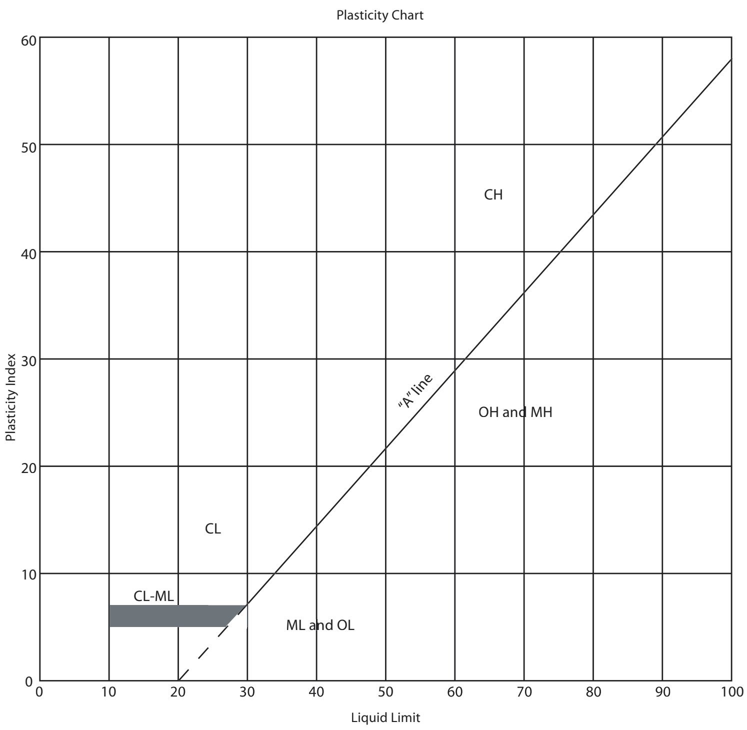
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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

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

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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

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

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information about This

Geotechnical-Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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