RED

Geotechnical Engineering Exploration and Analysis

Proposed Chick-fil-A Restaurant #4434
Silver Creek & Capital FSU
3095 Silver Creek Road
San Jose, California

Prepared for:

Chick-fil-A, Inc.
Irvine, California

Prepared by:

Giles Engineering Associates, Inc.

April 11, 2022
Project No. 2G-2108003

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ENGINEERING ASSOCIATES, INC.

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April 11, 2022

Chick-fil-A, Inc.
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Attention: Ms. Brenda Porrazzo
Senior Specialist, Strategy & Analytics

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Chick-fil-A Restaurant #4434
Silver Creek & Capital FSU
3095 Silver Creek Road
San Jose, California
Project No. 2G-2108003

Dear Ms. Porrazzo

Giles Engineering Associates, Inc. (Giles) is pleased to present our *Geotechnical Engineering Exploration and Analysis* report prepared for the above-referenced project. Conclusions and recommendations developed from the exploration and analysis are discussed in the accompanying report.

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

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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SAN JOSE, CALIFORNIA
PROJECT NO. 2G-2108003

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED CHICK-FIL-A RESTAURANT #4434
SILVER CREEK & CAPITAL FSU
3095 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA
PROJECT NO. 2G-2108003

1.0 EXECUTIVE SUMMARY OUTLINE

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

Subsurface Conditions

- Site Class designation D is recommended for seismic design considerations.
- Based on our review of the *Geologic Map of the San Jose East Quadrangle* (2000) compiled by United States Geological Survey indicated that the subject site is underlain by Holocene alluvial fan deposits and fine grained facies.
- Fill and possible fill soils encountered within our test borings at depths ranging from 3 ½ to 5 feet below grade were generally moist and consisted of firm to very stiff in comparative consistency lean clay, sandy silt, and silt with trace to little sand and firm in relative density silty sand and gravel.
- Native soils encountered beneath fill and possible fill soils within our test borings were generally moist to wet and consisted of firm to very stiff in comparative consistency lean clay, silty clay, sandy clay, sandy silt, and silt with trace sand and gravel and loose to firm in relative density clayey sand and gravel.
- Moist to very moist soil conditions were encountered within some of the near surface soils during our subsurface investigation. It is expected that similar conditions are likely to be encountered during grading operations. Grading operations may require blending of some of the drier soil and/or significant drying of the very moist soils prior to compaction and subgrade stabilization. Imported soils or base materials, as well as the use of lime treatment, may be required if onsite soils cannot be air-dried on site due to space, time constraints, or weather.
- Groundwater was encountered during our subsurface investigation at a depth of 34 feet below grade.
- Tested onsite soils generally possess a low expansion potential.
- Tested on-site soils have moderate corrosive potential when in contact with ferrous materials.

Site Development

- The proposed site development will include two phases (Phase 1 and Phase 2). Phase 1 will include the demolition of the existing building in the northern portion of the property for redevelopment as a parking lot. Phase 2 will include the construction of a new Chick-fil-A single-story building within the southern portion of the site and site improvements that will include drive-thru lane, new canopies, new parking stalls, menu board signs, a new trash enclosure, new concrete walkways, and new planter areas.

- Demolition of the existing building should include removal of all foundations, floor slabs, and any other below grade construction. Soils disturbed by the demolition operations should be removed and stockpiled for future use.
- New Building: Due to the variable strength characteristics of the near surface onsite soil and the likely disturbance of the soils during site clearing, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (5 feet minimum where feasible) be overexcavated to a depth of at least 2 foot below existing grade or planned pad grade, and at least 2 foot below bottom of footings, whichever is lower in elevation. The soils exposed at the base of this recommended over-excavation should be examined by the geotechnical engineer to document that the soils are suitable for building support. Depending on examination by the geotechnical engineer, deeper removals may be warranted. Prior to placement of fill, the exposed surface approved for fill placement should first be scarified to a depth of at least 6 to 8 inches, water conditioned or air dried as required to about 2 to 4 percent above the optimum moisture content and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00). A representative of the project geotechnical consultant should be present on site during grading operations to verify proper placement and adequate compaction of all fills.

Building Foundation

- The proposed structures may be supported by a shallow spread footing foundation system supported on structural compacted fill designed for a maximum, net allowable soil bearing pressure of 3,000 pounds per square foot (psf).
- Foundation reinforcement should be determined by the structural engineer.

Canopy Foundation

- Option 1: The proposed canopies may be supported by a shallow spread footing foundation system designed for a maximum, net allowable soil bearing pressure of 3,000 per square foot (psf) underlain by a minimum 2 foot structural fill layer.
- Option 2: The proposed canopies may be supported by drilled piers. In compacted fill, the piers may be designed for a maximum, net allowable soil bearing pressure of 3,000 psf plus a skin friction of 100 psf. For uplift resistance, an average allowable side resistance of 50 psf may be used for the piers.

Building Floor Slab

- It is recommended that an on grade slab be a minimum 4 inch thick slab-on-grade or turned-down slab, underlain by a minimum 4-inch thick granular base supported on a properly prepared subgrade.
- A minimum 15-mil vapor barrier is recommended to be directly below the floor slab or base course where required to protect moisture sensitive floor coverings.
- The floor is recommended to be designed as a mat on elastic subgrade based on a maximum modulus of subgrade reaction (ks) of 125 pci.

New Pavement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 8 or 10 inches of base course in parking stall and drive lane areas, respectively.
- Portland Cement Concrete: 6 inches in thickness underlain by 4 inches of base course in high stress areas such as entrance/exit aprons, drive-thru lane and the trash enclosure-loading zone.

Construction Considerations

- Any impacted soil associated with the former Tony's Unocal Self Service/O'Reilly's Auto Parts Store that was previously located in the Phase 1 construction area that are discovered during construction should be properly tested and disposed of off-site.

RED - This site has been given a Red designation due to some potential increased costs associated with the existing near surface moist to very moist clayey soils, and soils disturbed by building demolition.

2.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report. The scope of work performed for this report was consistent with the scope of work outlined within Proposal No. 2GEP-2107012.

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

Giles conducted a Phase I Environmental Site Assessment (ESA) for the subject site. The results of that assessment were provided under separate cover (2E-2108003).

3.0 SITES AND PROJECT DESCRIPTION

3.1 Site Description

A new Chick-fil-A restaurant is to be constructed at 3095 Silver Creek Road, in the City of San Jose, California. The Phase 1 construction area is occupied by a vacant building and the Phase 2 construction area (where the proposed Chick-fil-A will be located) is currently used as a parking lot.

The property is situated at approximately latitude 37.3076° North and longitude -121.8118° West. The existing parking lot appears to be in fair condition.

Based upon a review of the ALTA/NSPS land title survey prepared by PBLA Surveying, Inc., elevations at the proposed Chick-fil-A site are approximately El. 149 feet. The site is generally level.

3.2 Proposed Project Description

The proposed site development will include two phases (Phase 1 and Phase 2). Phase 1 will include the demolition of the existing building in the northern portion of the property for redevelopment as a parking lot. Phase 2 will include the construction of a new Chick-fil-A single-story building within the southern portion of the site. Although detailed building plans are not yet ready for our review, the new building will be a single-story wood-frame structure, 3,565 square feet, with no basement or underground levels. We were not provided with specific loading information for this project at the time of this report; however, based on previous experience with similar projects, we expect the maximum combined dead and live loads supported by the bearing walls and columns will be 2 to 3 kips per lineal foot (klf) and 40 to 50 kips, respectively. The live load supported by the floor slab is expected to be a maximum of 100 pounds per square foot (psf).

The footings for canopies are assumed to typically be 3 feet in diameter, about 5 feet in length, reinforced concrete caissons.

Other planned improvements include a drive thru, new parking lot and drive lanes, new canopies, menu board signs, concrete walkways and planter areas, and a trash enclosure.

According to the Preliminary Grading & Drainage Plan, prepared by Joseph C. Truxaw & Associates, sheet C3.0 and C3.1, dated February 9, 2022, the planned finish floor elevation for the proposed building will be at El. 150.00 feet. Therefore, site grading is anticipated to include minor cut and fill of up to 1 foot within the building area in order to establish the necessary site grade to accommodate the finished floor elevation exclusive of site preparation or over-excavation requirements necessary to create a stable site suited for the proposed development.

The traffic loading on the proposed parking lot improvement is understood to predominantly consist of automobiles with occasional heavy trucks resulting from deliveries and trash removal. The parking lot pavement sections have been designed on the basis of daily traffic intensity equivalent to five 18-kip single axle loads and 1,500 automobiles within the main drive lanes and only automobiles of a lesser intensity within the parking stalls. Pavement designs are based on a 20-year design period. Therefore, the parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.0 for the automobile traffic parking stalls (light duty) and a TI of 5.0 for drive lane areas (medium duty).

3.3 Background Information

The proposed Chick-fil-A site is a portion of a parking lot for a larger property and associated commercial building currently occupied by a Target, CVS Pharmacy, and Starbucks. The building and parking lot was first constructed in approximately 1975. Prior to the construction of the parking lot, the subject property and surrounding area was agricultural land.

The proposed parking area to the northwest of the proposed Chick-fil-A (Phase 1 construction area) is currently occupied by a vacant building. It was a former Tony's Unocal Self Service / former O'Reilly's Auto Parts Store and is listed on the UST, LUST, CUPA, and RCRA NON GEN databases. Three USTs were removed from the site in 1990 and two additional USTs were removed from the site in 2000, when the building was razed. A groundwater treatment system was in place between 1996 and 2004. Groundwater monitoring is currently ongoing at the site and petroleum hydrocarbons remain in soil and groundwater.

4.0 SUBSURFACE EXPLORATION

4.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of ten (10) test borings (B-1 to B-10) to depths of approximately 5 to 51 ½ feet below existing ground surfaces utilizing a truck rig with hollow-stem auger drilling equipment. The approximate test boring and percolation test locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes in accordance with ASTM D 3550, Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the borings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs with the number of blows required to drive the standard split-spoon sampler for the last 12 of the 18 inches reported. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

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

4.2 **Subsurface Conditions**

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

Site Geology

Based on our review of the *Geologic Map of the San Jose East Quadrangle* (2000) compiled by United States Geological Survey indicated that the subject site is underlain by Holocene alluvial fan deposits and fine grained facies.

Soil

Fill and possible fill soils encountered within our test borings at depths ranging from 3 ½ to 5 feet below grade were generally moist and consisted of firm to very stiff in comparative consistency lean clay, sandy silt, and silt with trace to little sand and firm in relative density silty sand and gravel. Possible deeper fills are expected within the northern proposed parking lot area within the area of the removed USTs.

Native soils encountered beneath fill and possible fill soils within our test borings were generally moist to wet and consisted of firm to very stiff in comparative consistency lean clay, silty clay, sandy clay, sandy silt, and silt with trace sand and gravel and loose to firm in relative density clayey sand and gravel.

Groundwater

Groundwater was encountered during our subsurface investigation at a depth of 34 feet below grade. However, fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

4.3 **Percolation Testing**

It is our understanding that an on-site below grade storm water infiltration system is being considered for the subject site. Therefore, two percolation tests were performed to assess the infiltration characteristics of the site soils in general accordance with USBR 7300-89.

The percolation testing consisted of drilling a four (4) inch diameter hole using a hollow-stem auger, installing a 2-inch-diameter slotted pvc casing with a solid end cap and then surrounding the casing with a granular filter pack. Clean pea gravel was used as the filter pack. The test holes (B-6 and B-10) were then pre-soaked to a minimum depth of 1 foot above the bottom of the boring and above the percolation test elevation. After pre-soaking, test water was added to the casing and refilled after each consecutive percolation test reading. The drop in water level over time is the pre-adjusted percolation rate at the test location. The pre-adjusted percolation rate is generally reduced to account for the discharge of water from both the sides and bottom of the boring. The formula below was used to calculate for the tested infiltration rate.

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

Where the reduction factor (R_f) is given by:

$R_f = (2di - \Delta d/ dia) + 1$

With: di = initial water depth (in.)

Δd = average/final water level drop (in.)

Dia = diameter of the boring (in.)

The results obtained from our percolation testing are summarized below.

TABLE 1 – PERCOLATION TEST RESULTS

Test Hole	Test Depth[1] (feet)	Pre-Adjusted Percolation Rate (in/hr)	Infiltration Rate (in/hr)	Soil Type
B-6	5.0	2.4	0.21	Lean Clay
B-10	5.0	3.36	0.51	Sandy Silt

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

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

5.0 LABORATORY TESTING

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

In Situ Moisture and Density

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

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Borings B-1 to B-4 (1 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829. The result of our expansion index (EI) test indicates that the near surface sample has a low expansion potential (EI = 35).

Sieve Analysis

Sieve Analyses (Passing No. 200 Sieve) was performed on selected samples from Test Borings B-1 and B-10 to assist in soil classification. This test was performed in accordance with Test Method ASTM D 1140. The results of the Passing No. 200 Sieve tests are presented on the Test Boring Log in Appendix A.

Atterberg Limits

The Atterberg Limits (liquid limit, plastic limit and plasticity index) were determined for representative samples of the on-site soils collected from Test Borings B-1, B-3, and B-6 in accordance with Test Method ASTM D 4318-00. The result of the Atterberg Limits is included on the Boring Logs enclosed in Appendix A. The near surface sample tested from Boring B-3 at 3 ½ feet had results of LL=42, PL=21, and PI=21.

Consolidation Test

Settlement and collapse/heave predictions under anticipated loads were made on the basis of a one-dimensional consolidation test. This test was performed in general accordance with Test Method ASTM D 2435 and D5333. The test sample was inundated at 2,000 psf pressure in order to evaluate the sudden increase in moisture condition (collapse potential). Result of this test indicated that the tested on-site soils exhibit a slight degree of collapse potential (0.3%). The Consolidation test curve, Figure 2, is included in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water soluble sulfate which could result in chemical attack of cement. These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The following table presents the results of our laboratory testing.

Parameter	**Bulk Sample 1 to 5 feet**
pH	7.2
Chloride	65 ppm
Sulfate	0.0011%
Resistivity	1,200 ohm-cm

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

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

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the concentrations present of water soluble sulfate which could result in chemical attack of cement. Our laboratory test data indicated that near surface soils contain approximately 0.0380 percent of water soluble sulfates. Based on Section 1904.1 of the 2016 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-11, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-11 a **negligible exposure to sulfate** can be expected for concrete placed in contact with the tested on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

6.0 GEOLOGIC AND SEISMIC HAZARDS

6.1 Active Fault Zones

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity.

6.2 Seismic Hazard Zones

Our review of the published Seismic Hazard Evaluation report for the San Jose East Quadrangle (where the subject site is located) indicates that the site is located in a zone of required investigation due to potential of earthquake induced liquefaction. In addition, historic high groundwater is reported to be approximately 20 feet below grade. Based on these conditions, a liquefaction analysis is deemed necessary.

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.

7.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

<u>Effect of Proposed Grading and Construction on Adjacent Property</u>

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

7.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed improvements should be designed in accordance with the current version of the 2019 California Building Code (CBC) and applicable local codes. Based upon the encountered subsurface soils and site geology, a Site Class D is recommended for design.

According to the maps of known active fault near-source zones (ICBO, 1998) to be used with the 2019 CBC, the Calaveras and Monte Vista-Shannon faults are the closest known active faults and are located about 5.75 and 6.89 miles, from the site, respectively. The Calaveras fault would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 7.03.

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

CBC 2019, Earthquake Loads	
Site Class Definition (Table 20.3-1)	D
Mapped Spectral Response Acceleration Parameter, S_s (for 0.2 second)	1.62
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.612
Site Coefficient, F_a short period	1.0
Site Coefficient, F_v 1-second period	1.7
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS}	1.62
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1}	1.04
Design Spectral Response Acceleration Parameter, S_{DS}	1.08
Design Spectral Response Acceleration Parameter, S_{D1}	0.693

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 $T_L \geq 1.5T_s$, or Equation (12.8-4) for $T > T_L$.

Liquefaction

Our review of the published Seismic Hazard Evaluation report for the San Jose East Quadrangle (where the subject site is located) indicates that the site is located in a zone of required investigation due to potential of earthquake induced liquefaction. In addition, historic high groundwater is reported to be approximately 20 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-1. The site acceleration (PGA_M) of 0.75g, was obtained from the SEAOC/OSHPD Seismic Design Map Tool and determined from ASCE 7-16. Predominant earthquake magnitude (Mw) at the site of 6.29 based upon a deaggregation analysis for a return period of 2,475 years was obtained from the USGS website. 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-1 @ 25 ft. [2]	36	18	21	0.58
B-1 @ 35 ft. [2]	32	14	15	0.47

[1]Potentially liquefiable
[2]Non-liquefiable.

The result of our analysis performed at boring B-1 is presented graphically as Plates A-1 of Appendix A. The computer output files are also included.

Based on the results of the liquefaction analysis (assumed high water of 20 feet), we estimate that ground settlement resulting from the design-level earthquake will be about 0.76 inches. In accordance with the seismically induced settlements required to be evaluated based upon ASCE 7-16 Sections 11.8 and 12.13.9, the calculated maximum differential settlement allowed per Table 12.13-3 Differential Settlement Threshold is 0.015(L) for Other Single-Story Structures (structures not consisting of concrete or masonry walls systems), where L is considered to occur over a total lateral distance of 30 feet. Therefore, the maximum allowable differential settlement is 5.4 inches (0.015 x 30 feet) over the 30 foot span, for seismically induced settlements, which is greater than the calculated 0.38 inches. Based on the results of our analysis, no mitigation is deemed necessary. The computer output files for the analysis are provided within Appendix A (A-1).

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.

7.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations and the results of our laboratory testing. **Moist to very moist soil conditions were encountered within the near surface on-site soils during our subsurface investigation. It is expected that similar conditions are likely to be encountered during grading operations. Grading operations may require significant provisions for drying, or blending of soils prior to compaction.** In addition, due to the presence of moist to very moist onsite soils at the proposed remedial grading depths, the loads imposed by heavy rubber-tired equipment during grading may induce localized pumping of the subgrade that would require stabilization prior to fill placement. Grading contractor should include contingencies for air-drying of excessively moist soil, as well as the stabilization of excavation bottoms in their bids. Imported soils, base materials or the use of lime treatment may be required if the soils cannot be air-dried on site due to space, time constraints, or weather conditions.

Site Clearing and Preparation

Clearing and demolition operations should include the removal of all landscape vegetation and any structural features such as building footings and floor slab, asphaltic concrete pavement, and concrete walkways within the area of the proposed new building and site improvements. If encountered, existing pavement within areas of proposed development should be removed or processed to a maximum 3-inch size and may be used as compacted fill or stabilizing material for the new development. Processed asphalt may be used as fill, sub-base course material, or subgrade stabilization material beyond the building perimeter. Processed concrete or existing base may be used as fill, sub-base course material, or subgrade stabilization material both within and outside of the building perimeter. Due to the moisture sensitivity and variable support characteristics of the on-site soils, the pavement is recommended to remain in-place as long as possible to help protect the subgrade from construction traffic disturbance.

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

Existing Utilities

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

Building Area

Due to the variable strength characteristics of the near surface onsite soils and the likely disturbance of the soils during site clearing, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (5 feet minimum where feasible) be overexcavated to a depth of at least 2 foot below existing grade or planned pad grade, and at least 2 foot below bottom of footings, whichever is lower in elevation. The soils exposed at the base of this recommended over-excavation should be examined by the geotechnical engineer to document that the soils are suitable for building support. Depending on examination by the geotechnical engineer, deeper removals may be warranted. Prior to placement of fill, the exposed surface approved for fill placement should first be scarified to a depth of at least 6 to 8 inches, water conditioned or air dried as required to about 2 to 4 percent above the optimum moisture content and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00). A representative of the project geotechnical consultant should be present on site during grading operations to verify proper placement and adequate compaction of all fills.

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

All utility trench backfill should be placed in lifts no greater than 12 inches in thickness, moisture conditioned and then compacted to a minimum of 90 percent of the soil’s maximum density near the optimum moisture content. A representative of the project geotechnical engineer should observe, probe, and test the backfills to document adequacy of compaction.

Proofroll and Compact Subgrades

Following site clearing, removal or re-compaction of disturbed soils and lowering of site grades where necessary for the 2 foot structural fill layer in the building area, the subgrades within the proposed building, pavement and drive through areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade, or stabilized in place. Depending on examination by the geotechnical engineer, some over-excavation as previously indicated may be required and should be budgeted accordingly. Any unsuitable materials discovered should be removed and backfilled with structural fill. Following proofrolling and completion of any necessary overexcavation, the subgrades in the parking lot and drive thru areas should be scarified to a depth of 6 to 8 inches, moisture conditioned above optimum moisture content and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557-00) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 90% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable low-expansive structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

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

A representative of the project geotechnical consultant should be present on site during grading operations to verify proper placement and adequate compaction of all fills.

Reuse of On-site Soil

On-site material may be reused as structural compacted fill (if needed) within the proposed building and pavement area provided they do not contain oversized materials and significant quantities of organic matter or other deleterious materials. Care should be used in controlling the moisture content of the soils to achieve proper compaction for load bearing. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering controlled conditions.

Subgrade Protection

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

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

Fill Placement

All fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned as required for the building pad and non-building pad areas, and then compacted to at least 90 or 95 percent of the Modified Proctor maximum density. A representative of the project geotechnical consultant should be present on-site during grading operations to document proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Import Structural Fill

Any soils imported to the site for use as structural fill should consist of very low to low expansive (EI less than 51) soils. Materials designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure, soluble sulfate content and corrosivity and pavement support characteristics.

7.3 Construction Considerations

Construction Dewatering

Groundwater was encountered during our subsurface investigation at a depth of 34 feet below grade. However, the site may be susceptible to a shallower perched water table due to seasonal precipitation and runoff characteristics of the site. Conventional filtered sump pumps placed in excavations are expected to be suitable for dewatering above the water table should any excess water conditions be observed.

Soil Excavation

Some localized slope stability problems may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Shallow excavations may be adequately sloped for bank stability while deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

Soil Disposal

Any impacted soil associated with the former Tony's Unocal Self Service/O'Reilly's Auto Parts Store that was previously located in the Phase 1 construction area that are discovered during construction should be properly tested and disposed of off-site.

7.4 Foundation Recommendations

Vertical Load Capacity

Upon completion of the recommended building pad preparation, it is our opinion the proposed structure may be supported by a shallow foundation system. Foundations may be designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf). Minimum foundation widths for walls and columns should be 18 and 24 inches, respectively, for bearing considerations, regardless of actual soil pressure. The maximum bearing value applies to combined dead and sustained live loads. This allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Canopy Drilled Pier/Footing Recommendations

For this foundation system embedded into compacted fill, or native material encountered within our borings verified in the field by the Geotechnical Engineer of Record during excavation, the axial (downward) skin friction (side resistance) resistance was determined to be 100 psf from our field data obtained during our site investigations at the site. This capacity is in addition to the allowable soil bearing pressure of 3,000 psf. We recommend a minimum pile spacing of 3 pier diameters with no reduction in axial capacity for group effects. The minimum recommended pile length is 5 feet.

Reduction to axial capacity loads as a result of downdrag forces is considered in the pier skin resistance capacity of 100 psf. Capacities for other pile types, dimensions, and lengths can be provided upon request.

For uplift resistance, an average allowable side resistance of 50 psf may be used for the piers.

It is recommended that a geotechnical engineer observe the drilled pier excavation procedures to confirm that the support soils are similar to those encountered at the test borings, and to confirm that the design parameters and estimated depths in the previous tables are representative of the actual subsurface conditions within the drilled pier excavations. If the design parameters are not appropriate for the actual conditions that are encountered, Giles must be contacted so that the design parameters in this report can be revised. Depending on the actual subsurface conditions within the pier excavations, the drilled piers might need to be wider and/or deeper than planned to adequately resist the proposed loads. The recommended soil design parameters are provided assuming that concrete for the drilled pier will be in direct contact with the surrounding soil.

General Drilled-Pier/Footing Construction Recommendations

Concrete should consist of a Portland cement mixture properly air-entrained, and with an appropriate water/cement ratio for proper strength and durability. Slump and maximum aggregate size must be selected so that the concrete will easily flow between reinforcing bars and will completely fill all voids.

It is recommended that a geotechnical engineer monitor the drilling operations to confirm that proper construction techniques are used. Strict safety precautions must be implemented and followed when near open excavations, such as pier excavations. An uncased pier excavation should not be approached, as it could rapidly cave. Concrete is recommended to be placed in accordance with "state-of-the-practice" procedures under engineering controlled conditions as noted below. Drilled pier construction should be done in accordance with local codes, and other pertinent requirements.

Pier/footing excavations should not be allowed to stand open, since a time delay could result in serious construction problems. A clean-out bucket should be used to remove disturbed soils within the drilled pier excavations. All bottom of excavations should be observed by the geotechnical engineer during drilling and prior to concrete placement to observe that all loose or disturbed soil has been removed.

Drilled Pier/Footing Lateral Loads

Resistance to lateral loads will be provided by the drilled piers. Active, At-Rest, and Passive Resistance (Equivalent Fluid Pressures) of 30 pcf, 45 pcf, and 350 pcf may be used for soil parameters, respectively. Reduction factors may be needed for group action for lateral capacities, dependent on the configuration of pier groups and the direction of applied lateral loads. The maximum recommended allowable passive pressure is 2,000 pcf.

Reinforcing

The reinforcement and design of the foundations and concrete sections should be performed by the project structural engineer.

Lateral Load Resistance

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

A coefficient of friction of 0.35 may be used with dead load forces for footings placed on compacted fill soil. An allowable passive earth pressure of 225 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against structural fill. The maximum recommended allowable passive pressure is 1,500 psf.

Bearing Material Criteria

Soil suitable to serve as the foundation structural fill subgrade should exhibit at least a loose relative density (average N value of at least 9) for non-cohesive soils, and an unconfined compressive strength of 1.5 tsf for cohesive soils, for the recommended 3,000 psf allowable soil bearing pressure. For design and construction estimating purposes, suitable soils to serve as the structural fill subgrade (native, existing fill, and possible fill) are expected to be encountered at nominal depths of 2 foot below the planned bearing grade following the recommended site preparation activities. However, field testing by the Geotechnical Engineer within the foundation structural fill subgrade soils is recommended to document that the foundation support soils possess the minimum strength parameters noted above. If unsuitable bearing soils are encountered, they should be recompacted in-place, if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade.

Foundation Embedment

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

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than ¾ and ½ inch, respectively. The estimated static differential movement is anticipated to result in an angular distortion of about 0.0014 inches per inch on the basis of a minimum clear span of 30 feet. The estimated seismic induced total and differential settlements is 0.76 and 0.38 inches, respectively. The combined static and seismic differential settlement is therefore estimated to be 0.51 inch over 20 feet. Therefore, resulting in an angular distortion of less than 0.0021 inches per inch, which is suitable for standard construction.

Pier Settlement Estimates and Considerations

Post-construction total and differential settlements of a pier foundation system designed in accordance with this report are estimated to be less than ⅔ and ⅓ inch, respectively. The angular distortion will be less than 0.0009 inch per inch across the planned span of 30 feet. The combined static and seismic differential settlement is estimated to be 0.0017 inch over 20 feet. The estimated settlements are considered within tolerable limits for the proposed structure provided they are appropriately considered in the structural design. Estimated settlements are based on the assumption that foundation support soil will be tested and approved by a geotechnical engineer and drilled pier construction will be observed by a geotechnical engineer during construction.

Giles should review the final approved design/plans prior to construction.

7.5 Floor Slab Recommendations

Subgrade

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

Design

The floor of the proposed building may be designed and constructed as a conventional slab-on-grade supported on a properly prepared subgrade. If desired, the floor slab may be poured monolithically with perimeter foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. The floor slab is recommended to be designed based on a maximum modulus of subgrade reaction (k_s) of 125 pci. The slab is recommended to be a minimum of 4 inches in thickness. A qualified structural engineer should perform the actual design of the slab to ensure proper thickness and reinforcing.

A minimum 6-inch-thick base course is recommended to be directly below the vapor retarder to serve as a capillary break, which may be part of the 2 foot structural fill layer. It is recommended that the base course consist of free-draining aggregate. Also, it is recommended that a geotechnical engineer test and approve base-course aggregate before it is placed.

A vapor barrier should be incorporated into the floor slab design in all areas where moisture-sensitive floor coverings, coatings, underlayments, adhesives, moisture sensitive goods, humidity-controlled environments, or climate-cooled environments are anticipated initially, or in the future. Vapor barrier should consist of a minimum 15 mil extruded polyolefin plastic (no recycled content or woven materials permitted); permeance as tested before and after mandatory conditioning (ASTM E1745 section 7.1 and sub-paragraphs 7.1.1 – 7.1.5): less than 0.01 perms [grains/($ft^2 \cdot hr \cdot inhg$)] and comply with the ASTM E1745 class a requirements. The vapor barrier should also meet paragraph's 8.1 and 9.3 of ASTM E1745; subsequent documentation should be provided by the vapor barrier manufacturer. Install vapor barrier in accordance with ASTM E1643, including proper perimeter seal. An additional 2-inch thick layer of coarse sand may be needed between the slab and the vapor retarder to promote proper curing. The sand layers above and below the synthetic sheeting may be used as a substitute for the granular material below the slab. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Settlement

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

7.6 New Pavement

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

Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of soil that exhibits a low expansion potential. An R-value of 5 has been assumed in the preparation of the pavement design. It should however, be recognized that the local municipality may require a specific R-value test to verify the use of the following design. It is recommended that this testing, if required, be conducted following completion of rough grading in the proposed pavement areas so that the R-value test results are indicative of the actual pavement subgrade soils. Alternatively, a minimum code pavement section may be required if a specific R-value test is not performed. To use this R-value, all fill added to the pavement subgrade must have pavement support characteristics at least equivalent to the existing soils, and must be placed and compacted in accordance with the project specifications.

Asphalt Pavements

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

ASPHALT PAVEMENTS			
Materials	**Thickness (inches)**		**CALTRANS Specifications**
	Parking Stalls (TI=4.0)	**Drive Lanes (TI=5.0)**	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	8	10	Section 26, Class 2 (R-value at least 78)
NOTES: (a) Compaction to density between 95 and 100 percent of the 50-Blow Marshall Density (b) The surface and binder course may be combined as a single layer placed in one lift if similar materials are utilized.			

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

Portland Concrete Pavements

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

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

General Considerations

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

7.7 Recommended Construction Materials Testing Services

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

7.8 Basis of Report

This report is based on Giles' proposal, which is dated August 2, 2021 and is referenced by Giles' proposal number 2GEP-2107012. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

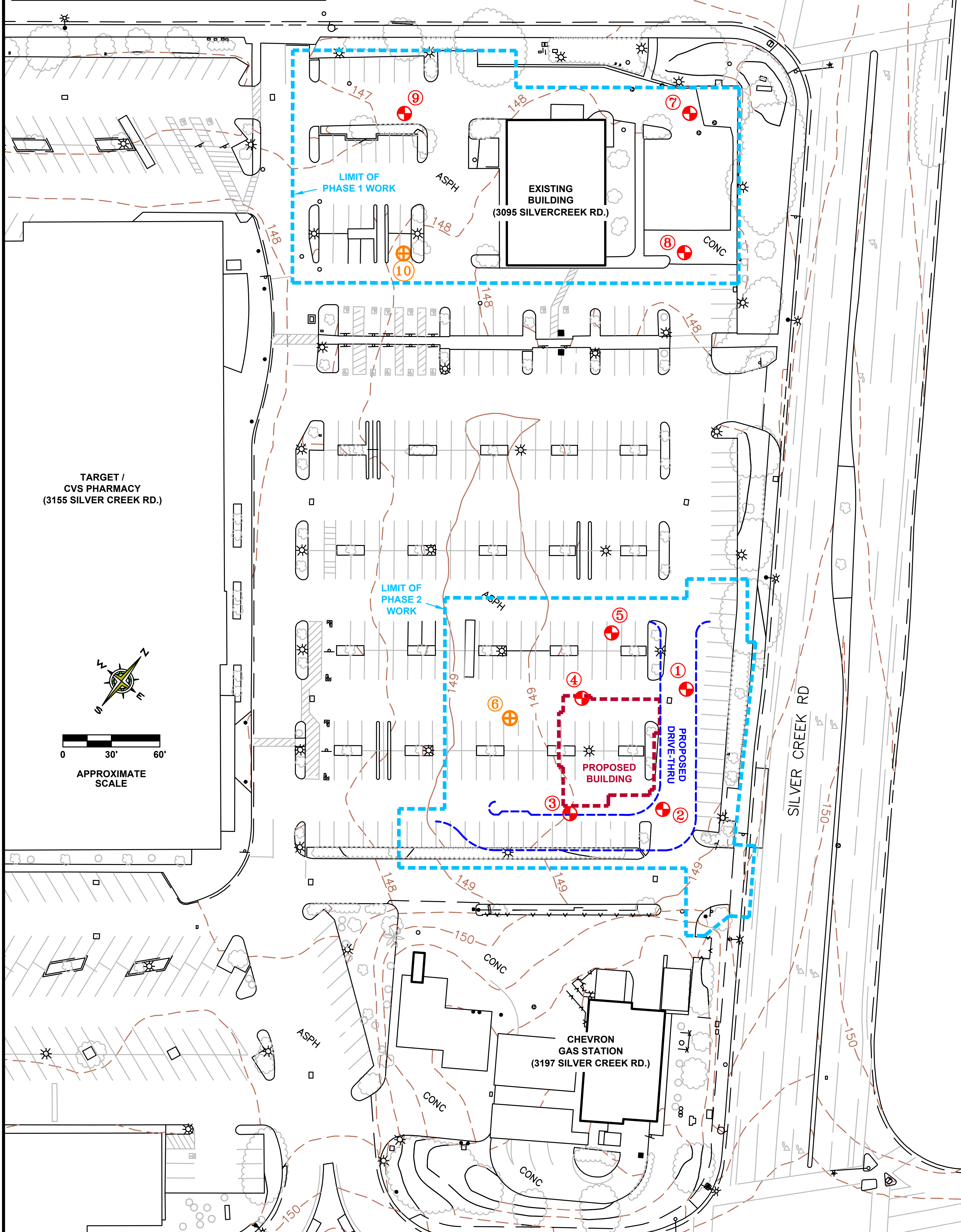
The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were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

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "ALTA / NSPS SURVEY", REV. 4-3-2019, PREPARED BY PBLA SURVEYING, INC.

3.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "SITE PLAN EXHIBIT" (SHEET G4.0), REV. 6-21-2021, PREPARED BY WARE MALCOMB.



GILES ENGINEERING ASSOCIATES, INC.
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FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 04434
SILVER CREEK & CAPITOL FSU
3095 SILVER CREEK ROAD
SAN JOSE, CALIFORNIA

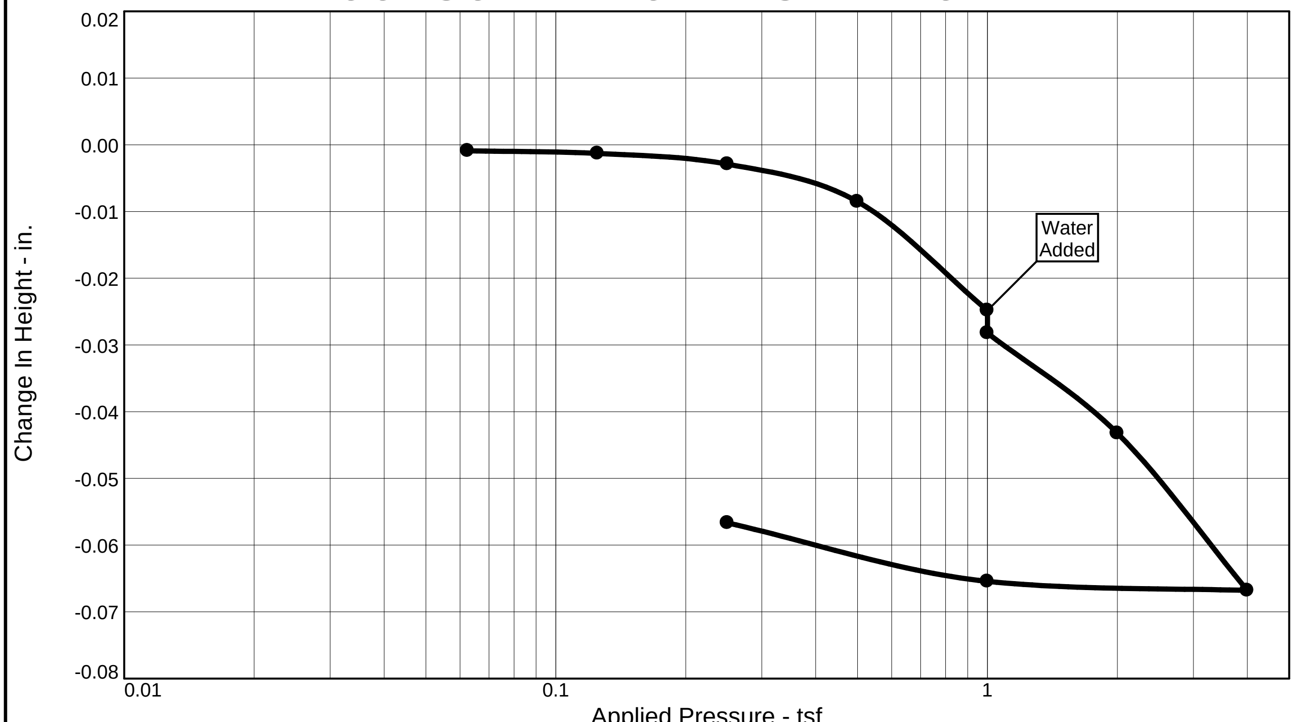
DESIGNED	DRAWN	SCALE	DATE	REVISED
MLS	[signature]	approx. 1"=60'	12-03-21	--
PROJECT NO.: 2G-2108003			CAD No. 2g2108003-blp	

LEGEND:

① GEOTECHNICAL TEST BORING

⑥ GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
8	4.00	2.557									
9	1.00	1.155									
10	0.25	0.403									

MATERIAL DESCRIPTION	USCS	AASHTO

LL	PI	Sp. Gr.	Overburden (tsf)	Dry Dens. (pcf) Init.	Dry Dens. (pcf) Final	Moisture Init.	Moisture Final	Saturation Init.	Saturation Final	Void Ratio Init.	Void Ratio Final	P_c (tsf)	C_c
		2.7		107.7	116.1	17.5 %	20.4 %	83.8 %	100.0 %	0.565	0.477	2.1	0.12

Preparation Process: Trimmed with a wire saw

Condition of Test: Good

D2435 Method	C_s	Swell Press. (tsf)	Clpse. %
B	0.01		0.3

Project No. 2G-2108003 **Client:** Chick-Fil-A (Atlanta, GA)

Project: Chick-Fil-A No. 3435
Silver Creek San Jose, CA

Source of Sample: Boring No. 4 **Depth:** 3½ ft.

GILES ENGINEERING ASSOC., INC.

Waukesha, Wisconsin

Remarks:

Checked By:

Title:

Figure

BORING NO. & LOCATION:	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
B- 1		
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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			1-SS	15		4.5+		21		
Dark Gray lean Clay, trace Sand - Very Moist (Possible Fill)			2-SS	12		3.5		19		
Dark Brown Sandy Silt - Moist			3-SS	11		4.25		16		
Brown Silt, trace Sand - Moist		140								
Gray and Brown Silty Clay - Moist	10		4-SS	8				21		
Gray Sandy Clay - Moist			5-SS	12		3.5		21		
		130								
Dark Gray lean Clay - Moist	20		6-SS	14				26		
Brown lean Clay - Moist			7-SS	11		4.0		21		LL=36 PL=18 PI=18
		120								
Brown lean Clay, some small Gravel - Moist	30		8-SS	27		4.5+		16		
Brown lean Clay, little Sand and Gravel - Very Moist			9-SS	31		4.5+		15		LL=32 PL=18 PI=14
		110								
Brown Clayey Sand and Gravel - Very Moist	40		10-SS	27				8		
Brown Clayey fine to coarse Sand - Wet			11-SS	29				22		P_{200}=26%
Boring Terminated at about 46.5 feet (EL. 102.5')										

Water Observation Data	Remarks:
Water Encountered During Drilling: 34 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- 2	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of asphaltic concrete over 5 inches of aggregate base										
Dark Gray lean Clay, trace Sand - Very Moist (Possible Fill)			1-SS	13		4.25		26		
Dark Brown Gray lean Clay, trace Sand - Moist	5	145	2-CS	24				21		Dd=95.7 pcf
Dark Brown Sandy Silt, trace Clay - Moist			3-CS	25		3.5		17		Dd=103.0 pcf
	10	140								
Brown Silty Clay, trace Sand - Moist			4-SS	10				23		
	15	135								
Gray and Brown lean Clay, trace Sand - Moist			5-SS	11				16		

Boring Terminated at about 16.5 feet (EL. 132.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: None	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

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

GILES LOG REPORT 2G-2108003.GPJ GILES.GDT 12/21/21

BORING NO. & LOCATION:	TEST BORING LOG	
B- 3		
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 Sand, and Gravel - Moist (Fill)			1-SS	15				5		
Dark Gray lean Clay, trace Sand - Moist (Possible Fill)	5	145	2-SS	11		4.5+		19		LL=42 PL=21 PI=21
Brown Silty Clay, trace Sand - Moist		140	3-SS	12		4.5		17		
Brown Silty fine Sand - Moist	10	135	4-SS	8				23		
Brown and Gray lean Clay - Moist	15		5-SS	9		3.25		22		

Boring Terminated at about 16.5 feet (EL. 132.5')

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

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

BORING NO. & LOCATION: B- 4	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 7 inches of aggregate base										
Dark Gray lean Clay, trace Sand - Moist (Possible Fill)			1-SS	14		4.5+		23		
Dark Gray Silt, trace Sand - Moist (Possible Fill)	5	145	2-CS	18				17		Dd=107.7 pcf
Dark Brown Silty Clay, trace Sand - Moist		140	3-CS	21				18		Dd=100.4 pcf
Brown Silty Clay - Moist	10	135	4-SS	7				23		
Brown lean Clay - Moist	15		5-SS	7		3.25		20		

Boring Terminated at about 16.5 feet (EL. 132.5')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: None	CS = California Split Spoon
▼	Water Level At End of Drilling:	SS = Standard Penetration Test
	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
	Cave Depth After Drilling:	

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

BORING NO. & LOCATION:	TEST BORING LOG	
B- 5		
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 7 inches of aggregate base										
Dark Gray lean Clay, little Sand - Very Moist (Possible Fill)		147.5	1-SS	16		4.0		26		
	2.5									
Dark Gray lean Clay, little Sand - Moist (Possible Fill)		145.0	2-SS	10		4.5+		19		
	5.0									

Boring Terminated at about 5 feet (EL. 144')

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

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

BORING NO. & LOCATION: B- 6	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 149 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 8 inches of aggregate base										
Dark Gray lean Clay, trace Sand - Moist (Possible Fill)		147.5	1-SS	8		4.25		23		
	2.5									
Gray lean Clay - Moist		145.0	2-SS	12		4.5+		18		LL=45 PL=21 PI=24
	5.0									
Boring Terminated at about 5 feet (EL. 144')										

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.

GILES LOG REPORT 2G-2108003.GPJ GILES.GDT 12/21/21

BORING NO. & LOCATION: B- 7	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 148 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 7 inches of aggregate base		147.5								
Olive Brown Silty Sand, trace Gravel - Moist (Fill)	2.5	145.0	1-SS	15				27		
Brown lean Clay - Moist (Possible Fill)	5.0		2-SS	12		4.5		19		
Boring Terminated at about 5 feet (EL. 143')										

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

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

BORING NO. & LOCATION: B- 8	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 148 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 5 inches of aggregate base		147.5								
Dark Gray lean Clay, trace Sand - Very Moist (Possible Fill)	2.5	145.0	1-SS	11		4.25		27		
Dark Brown to Brown lean Clay, trace Sand and Gravel - Moist	5.0		2-SS	8		4.0		20		
Boring Terminated at about 5 feet (EL. 143')										

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

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

BORING NO. & LOCATION: B- 9	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 147 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 7 inches of aggregate base										
Brown lean Clay - Moist (Possible Fill)		145.0	1-SS	14		4.5+		25		
	2.5									
Brown lean Clay - Moist		142.5	2-SS	19		4.5+		19		
	5.0									
Boring Terminated at about 5 feet (EL. 142')										

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

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

BORING NO. & LOCATION: B-10	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 148 feet	PROPOSED CHICK-FIL-A RESTAURANT #4434	
COMPLETION DATE: 09/28/21	3095 SILVER CREEK ROAD SAN JOSE, CA	
FIELD REP: ROBERT TORRES	PROJECT NO: 2G-2108003	

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 7 inches of aggregate base		147.5								
Dark Brown to Brown Sandy Silt, trace Clay and Gravel - Moist (Fill)			1-SS	10		3.5		21		
	2.5	145.0								
Dark Brown Sandy Silt- Moist			2-SS	15		4.5+		19		P_{200}=56%
Boring Terminated at about 5 feet (EL. 143')	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:	

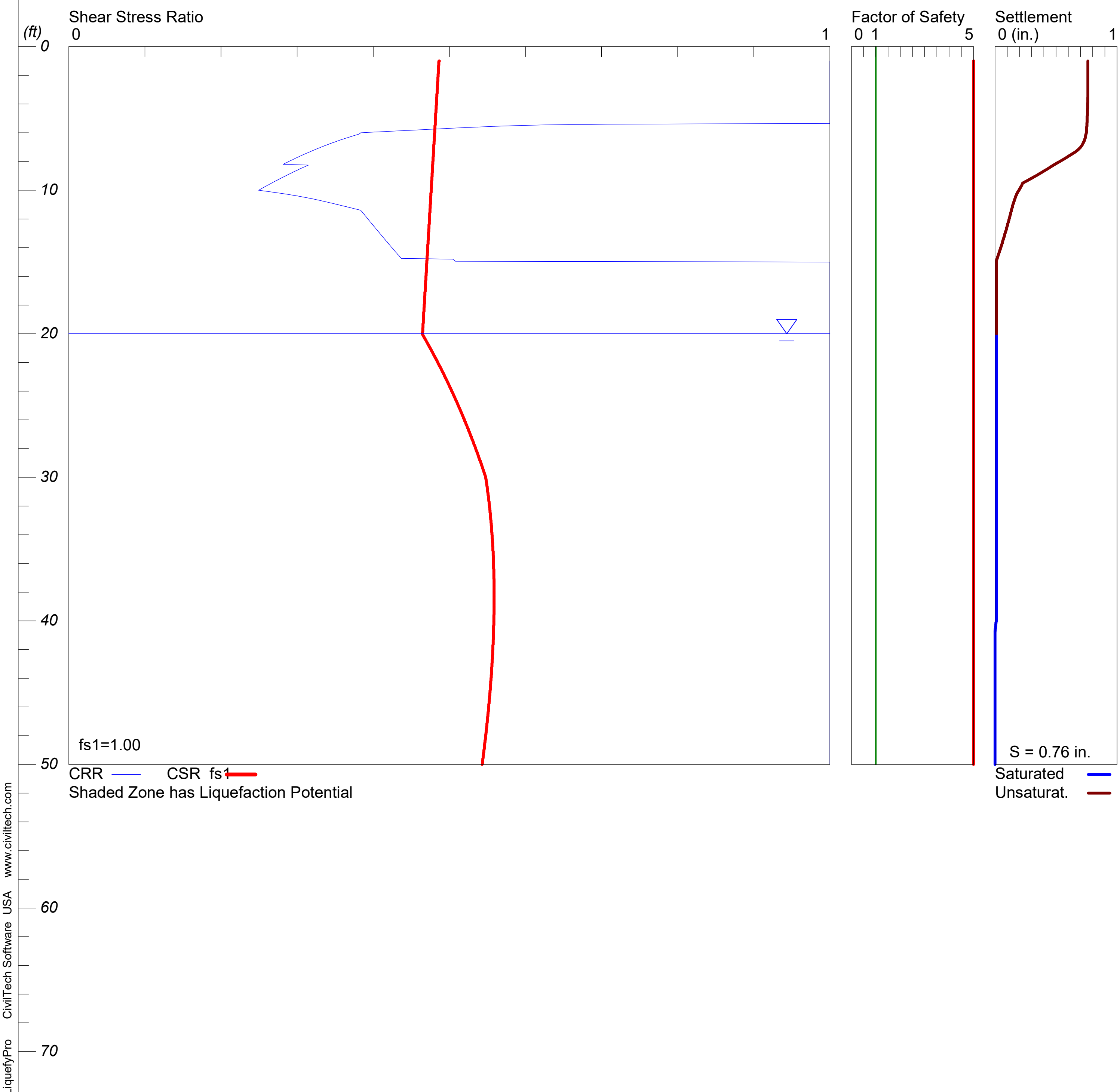
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

CFA San Jose, CA #4434

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

Magnitude=6.29
Acceleration=0.75g



CivilTech Corporation

Plate A-1

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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: P:\John Maier\Chick-fil-A\Geo\San Jose 4434\B-1.liq
Title: CFA San Jose, CA #4434
Subtitle:

Surface Elev.=
Hole No.=B-1
Depth of Hole= 50.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 34.00 ft
Max. Acceleration= 0.75 g
Earthquake Magnitude= 6.29

Input Data:

Surface Elev.=
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 20.00 ft
Water Table during In-Situ Testing= 34.00 ft
Max. Acceleration=0.75 g
Earthquake Magnitude=6.29
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

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

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
1.00	15.00	120.00	NoLiq
3.50	12.00	120.00	NoLiq
6.00	11.00	120.00	10.00
10.00	8.00	120.00	10.00
15.00	12.00	120.00	NoLiq
20.00	14.00	120.00	NoLiq
25.00	11.00	120.00	NoLiq
30.00	27.00	120.00	NoLiq
35.00	31.00	120.00	NoLiq
40.00	27.00	120.00	26.00
45.00	29.00	120.00	40.00
50.00	29.00	120.00	40.00

Output Results:

Settlement of Saturated Sands=0.01 in.
Settlement of Unsaturated Sands=0.75 in.
Total Settlement of Saturated and Unsaturated Sands=0.76 in.
Differential Settlement=0.381 to 0.503 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
1.00	2.00	0.49	5.00	0.01	0.75	0.76
1.50	2.00	0.49	5.00	0.01	0.75	0.76
2.00	2.00	0.49	5.00	0.01	0.75	0.76
2.50	2.00	0.48	5.00	0.01	0.75	0.76
3.00	2.00	0.48	5.00	0.01	0.75	0.76
3.50	2.00	0.48	5.00	0.01	0.75	0.76
4.00	3.14	0.48	5.00	0.01	0.75	0.76
4.50	3.14	0.48	5.00	0.01	0.75	0.76
5.00	3.14	0.48	5.00	0.01	0.74	0.76
5.50	0.59	0.48	5.00	0.01	0.74	0.75
6.00	0.38	0.48	5.00	0.01	0.74	0.75
6.50	0.36	0.48	5.00	0.01	0.72	0.73
7.00	0.33	0.48	5.00	0.01	0.69	0.70
7.50	0.31	0.48	5.00	0.01	0.62	0.63
8.00	0.29	0.48	5.00	0.01	0.52	0.53
8.50	0.30	0.48	5.00	0.01	0.42	0.43
9.00	0.28	0.48	5.00	0.01	0.32	0.33
9.50	0.27	0.48	5.00	0.01	0.22	0.23
10.00	0.25	0.48	5.00	0.01	0.18	0.20
10.50	0.31	0.48	5.00	0.01	0.16	0.17

11.00	0.35	0.47	5.00	0.01	0.14	0.15
11.50	0.39	0.47	5.00	0.01	0.12	0.13
12.00	0.39	0.47	5.00	0.01	0.10	0.12
12.50	0.40	0.47	5.00	0.01	0.09	0.10
13.00	0.41	0.47	5.00	0.01	0.07	0.08
13.50	0.42	0.47	5.00	0.01	0.05	0.07
14.00	0.42	0.47	5.00	0.01	0.04	0.05
14.50	0.43	0.47	5.00	0.01	0.02	0.03
15.00	2.00	0.47	5.00	0.01	0.00	0.01
15.50	2.00	0.47	5.00	0.01	0.00	0.01
16.00	2.00	0.47	5.00	0.01	0.00	0.01
16.50	2.00	0.47	5.00	0.01	0.00	0.01
17.00	2.00	0.47	5.00	0.01	0.00	0.01
17.50	2.00	0.47	5.00	0.01	0.00	0.01
18.00	2.00	0.47	5.00	0.01	0.00	0.01
18.50	2.00	0.47	5.00	0.01	0.00	0.01
19.00	2.00	0.47	5.00	0.01	0.00	0.01
19.50	2.00	0.47	5.00	0.01	0.00	0.01
20.00	2.00	0.46	5.00	0.01	0.00	0.01
20.50	2.00	0.47	5.00	0.01	0.00	0.01
21.00	2.00	0.47	5.00	0.01	0.00	0.01
21.50	2.00	0.48	5.00	0.01	0.00	0.01
22.00	2.00	0.48	5.00	0.01	0.00	0.01
22.50	2.00	0.49	5.00	0.01	0.00	0.01
23.00	2.00	0.49	5.00	0.01	0.00	0.01
23.50	2.00	0.50	5.00	0.01	0.00	0.01
24.00	2.00	0.50	5.00	0.01	0.00	0.01
24.50	2.00	0.51	5.00	0.01	0.00	0.01
25.00	2.00	0.51	5.00	0.01	0.00	0.01
25.50	2.00	0.52	5.00	0.01	0.00	0.01
26.00	2.00	0.52	5.00	0.01	0.00	0.01
26.50	2.00	0.52	5.00	0.01	0.00	0.01
27.00	2.00	0.53	5.00	0.01	0.00	0.01
27.50	2.00	0.53	5.00	0.01	0.00	0.01
28.00	2.00	0.53	5.00	0.01	0.00	0.01
28.50	2.00	0.54	5.00	0.01	0.00	0.01
29.00	2.00	0.54	5.00	0.01	0.00	0.01
29.50	2.00	0.54	5.00	0.01	0.00	0.01
30.00	2.00	0.55	5.00	0.01	0.00	0.01
30.50	2.00	0.55	5.00	0.01	0.00	0.01
31.00	2.00	0.55	5.00	0.01	0.00	0.01
31.50	2.00	0.55	5.00	0.01	0.00	0.01
32.00	2.00	0.55	5.00	0.01	0.00	0.01
32.50	2.00	0.55	5.00	0.01	0.00	0.01
33.00	2.00	0.55	5.00	0.01	0.00	0.01
33.50	2.00	0.56	5.00	0.01	0.00	0.01
34.00	2.00	0.56	5.00	0.01	0.00	0.01
34.50	2.00	0.56	5.00	0.01	0.00	0.01

35.00	2.00	0.56	5.00	0.01	0.00	0.01
35.50	3.02	0.56	5.00	0.01	0.00	0.01
36.00	3.02	0.56	5.00	0.01	0.00	0.01
36.50	3.01	0.56	5.00	0.01	0.00	0.01
37.00	3.01	0.56	5.00	0.01	0.00	0.01
37.50	3.01	0.56	5.00	0.01	0.00	0.01
38.00	3.00	0.56	5.00	0.01	0.00	0.01
38.50	3.00	0.56	5.00	0.01	0.00	0.01
39.00	2.99	0.56	5.00	0.01	0.00	0.01
39.50	2.99	0.56	5.00	0.01	0.00	0.01
40.00	2.99	0.56	5.00	0.01	0.00	0.01
40.50	2.98	0.56	5.00	0.00	0.00	0.00
41.00	2.98	0.56	5.00	0.00	0.00	0.00
41.50	2.97	0.56	5.00	0.00	0.00	0.00
42.00	2.97	0.56	5.00	0.00	0.00	0.00
42.50	2.97	0.56	5.00	0.00	0.00	0.00
43.00	2.96	0.56	5.00	0.00	0.00	0.00
43.50	2.96	0.56	5.00	0.00	0.00	0.00
44.00	2.96	0.55	5.00	0.00	0.00	0.00
44.50	2.95	0.55	5.00	0.00	0.00	0.00
45.00	2.95	0.55	5.00	0.00	0.00	0.00
45.50	2.94	0.55	5.00	0.00	0.00	0.00
46.00	2.94	0.55	5.00	0.00	0.00	0.00
46.50	2.94	0.55	5.00	0.00	0.00	0.00
47.00	2.93	0.55	5.00	0.00	0.00	0.00
47.50	2.93	0.55	5.00	0.00	0.00	0.00
48.00	2.93	0.55	5.00	0.00	0.00	0.00
48.50	2.92	0.55	5.00	0.00	0.00	0.00
49.00	2.92	0.55	5.00	0.00	0.00	0.00
49.50	2.91	0.54	5.00	0.00	0.00	0.00
50.00	2.91	0.54	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.

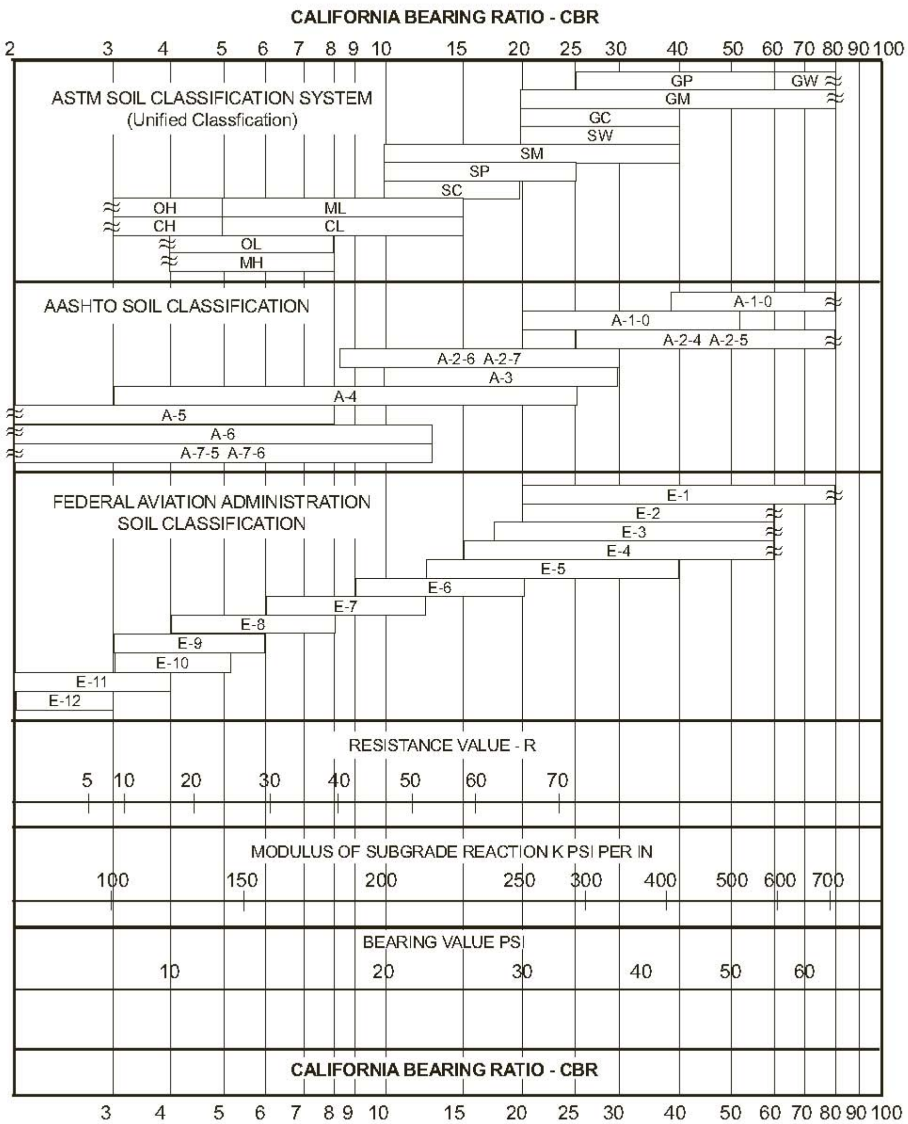
Laboratory Testing

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.

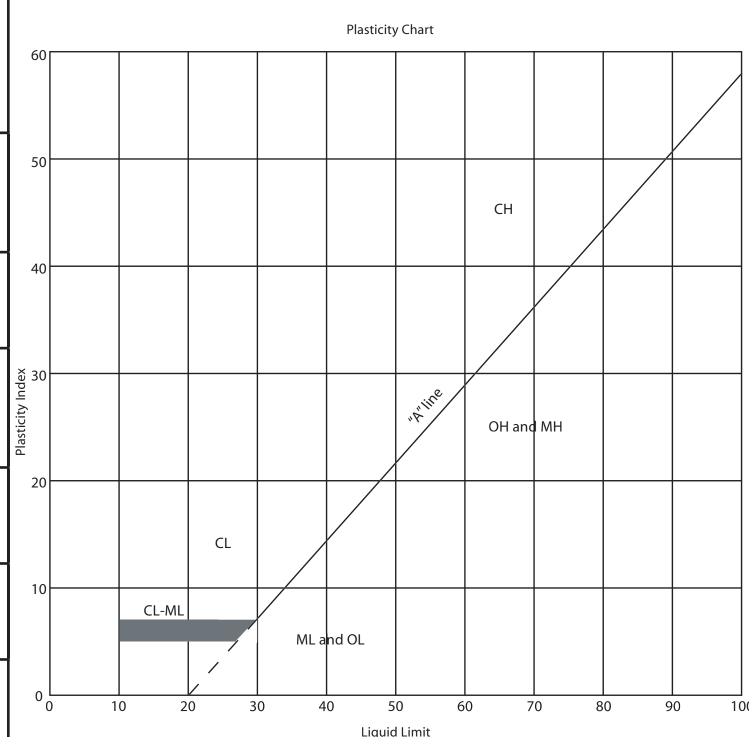
The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

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, mica-ceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information about 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 <u>not</u> 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 <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> 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 <u>not</u> rely on an executive summary. Do <u>not</u> 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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

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