GREEN

Geotechnical Engineering Exploration and Analysis

Proposed Chick-fil-A Restaurant #4306
I-5 and Palomar FSU
5850 Avenida Encinas
Carlsbad, California

Prepared for:

Chick-fil-A, Inc.
Irvine, California

Prepared by:

Giles Engineering Associates, Inc.

March 14, 2019
Project No. 2G-1808005

GILES
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GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

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March 14, 2019

Chick-fil-A, Inc.
15635 Alton Parkway, Suite 350
Irvine, California 92618

Attention: Ms. Beth Witt
Development Coordinator

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Chick-fil-A Restaurant #4306
I-5 and Palomar FSU
5850 Avenida Encinas
Carlsbad, California
Project No. 2G-1808005

Dear Ms. Witt:

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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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

CHICK-FIL-A RESTAURANT #4306
I-5 AND PALOMAR FSU
5850 AVENIDA ENCINAS
CARLSBAD, CALIFORNIA
PROJECT NO. 2G-1808005

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.
- Our review of the Geology of San Diego Quadrangle indicates that the site is mapped as being underlain by Old Paralic Deposits consisting generally of poorly sorted, moderately permeable, reddish-brown, interfingered strand like, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate.
- Possible fills were encountered within our test borings to depths of about 3 feet below existing ground surfaces and were noted to be moist, medium dense in relative density clayey sand and silty sand, and firm in comparative consistency sandy clay.
- Native soils encountered below the possible fills were generally moist, medium dense to dense silty sand and sand, and very stiff sandy clay.
- Old Paralic Deposits were encountered within test borings B-1 and B-4 to depths of about 18 to 20 feet below existing ground surface and generally consisted of very dense silty sandstone materials.
- Groundwater was encountered during our subsurface exploration to a depth of about 17 and 18 feet below existing grade within test borings B-1 and B-4.

Site Development

- The proposed site development will include the demolition of existing building for the construction of a new Chick-fil-A single-story building and site improvements that include new concrete walkways, parking stalls, driveways, drive thru lane, and trash enclosure.
- Building Area: Due to the presence of variable strength characteristics of the near surface soils and likely disturbance of site soils during clearing operations, it is recommended that the soils within the proposed new building and an appropriate distance beyond (5 feet minimum) be over-excavated to a depth of at least 2 feet below existing grade or planned grade and 1 foot below bottom of footings, whichever is greater. 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. Prior to placement of fill, the exposed surfaces approved for fill placement should be scarified to a depth of at least 12 inches, moisture conditioned and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557-00).
- Due to the presence of dense to very dense onsite soils some excavation difficulties should be expected.

Building Foundation

- Shallow spread footing foundation systems or turned-down slabs may be designed for a maximum, net allowable soil pressure of 3,000 psf soil bearing pressure supported on newly placed structural compacted fill.
- Minimum reinforcing in the strip footings is recommended to consist of four No. 5 bars (2 top and 2 bottom).

Building Floor Slab

- It is recommended that on grade slab be a minimum 4-inch thick slab-on-grade or turned-down slab, underlain by properly prepared subgrade.
- Minimum slab reinforcing recommended consisting of No. 3 rebars spaced at 18 inches on center, each way.

Parking Improvement

- Asphalt Pavements: 3 inches of asphaltic concrete underlain by 5 and 8 inches of base course aggregate in parking stalls and driveways, respectively.
- Portland Cement Concrete: 6 inches in thickness underlain by 4 inches of base course in high stress areas such as entrance/exit aprons, trash enclosure-loading zone, and the drive through area.

GREEN - This site has been given a Green designation to indicate that there are no significant geotechnical related construction or recognized problems foreseen which are unusual or not typical to this general area.

2.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report.

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

Giles conducted a *Phase 1 Environmental Site Assessment* for the subject site. The results of that assessment were provided under separate cover (2E-1808009).

3.0 SITES AND PROJECT DESCRIPTION

3.1 Site Description

The proposed Chick-fil-A site is currently an active two-story office building, about 10,977 square feet, and located at 5850 Avenida Encinas, in the city of Carlsbad, California.. The roughly triangular shaped property is bounded on the north and west by Avenida Encinas, on the south by In-N-Out restaurant, and on the east by the I-5 freeway. The existing building is situated within the central portion of the site and bordered with parking stalls and drive ways to the north, east and south sides, and landscape area to the west by Avenida Encinas.
.
Based upon a review of the ALTA/NSPS Land Title Survey prepared by Joseph Truxaw and Associates, elevations at the site range from El. 56 feet to El. 58 feet. The site is relatively level and slopes to the northwest by the adjacent street (Avenida Encinas). The subject property is situated at approximately latitude of 33.1255^{o} North and longitude of -117.3247^{o} West.

The site is currently covered with asphalt pavement, curbs and few landscape planters that contain shrubs and trees. Other existing site improvements include asphalt pavement along with curbs and gutter, concrete v-gutter, concrete walkways, lighting poles, chain linked fence, trash enclosure, landscape areas containing grass, shrubs and trees, and underground utilities.

3.2 Proposed Project Description

The proposed development includes the demolition of existing building for the construction of a new, single-story Chick-fil-A restaurant building with drive through lane to be located along the southeasterly portion of the site adjacent to I-5 freeway and within a portion of the southerly side of the existing building (Figure 1). The drive through lane will be located to the northerly side of the new building. The new building will be a single-story wood-frame structure, 3,201 square feet, with no basement or underground levels to be located within the northern end of the property. We were not provided with specific loading information for this project at the time of this report; however, based on previous Chick-fil-A projects, we expect maximum combined dead and live loads supported by the bearing walls and columns of 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).

Other planned improvements include new parking lot, menu board signs, outdoor dining area, a playground area, concrete walkways and planter areas, and a trash enclosure. Parking lot improvement within the property will include curbs and gutters, and underground utilities. Three bio-filtration basins are planned at the site and these basins will be located more than 20 feet away from the new CFA building.

According to the Conceptual Grading Plan, prepared by Joseph C. Truxaw & Associates, dated March 1, 2019, the planned finish floor elevation for the proposed building will be at El. 57.85 feet. Therefore, site grading will consist of minor cut and fill (less than 1 foot) in order to establish the necessary site grade to accommodate the planned 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).

4.0 SUBSURFACE EXPLORATION

4.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of six (6) exploratory test borings to depths of about 5 to 35½ feet below existing ground surfaces. The approximate test boring 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 and results are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the borings. Relatively undisturbed samples were collected (per ASTM D-3550) using a 3-inch outside-diameter, modified California split-spoon soil sampler (CS) lined with 1-inch high brass rings. The sampler was driven with successive 30-inch drops of a hydraulically operated, 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the field exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

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

4.2 Subsurface Conditions

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

Pavement

Existing pavement encountered within our test borings consisted of approximately 2½ to 5 inches thick asphalt concrete over 4½ to 5 inches of aggregate base. No aggregate base was noted within test borings B-2, B-3 and B-4. Based on our visual observation, the existing asphalt pavement is in fair to poor condition.

Soil

Our review of the Geology of San Diego Quadrangle indicates that the site is mapped as being underlain by Old Paralic Deposits consisting generally of poorly sorted, moderately permeable, reddish-brown, interfingered strand like, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate.

Possible fills were encountered within our test borings to depths of about 3 feet below existing ground surfaces and were noted to be moist, medium dense in relative density clayey sand and silty sand, and firm in comparative consistency sandy clay.

Native soils encountered below the possible fills were generally moist, medium dense to dense silty sand and sand, and very stiff sandy clay.

Old Paralic Deposits were encountered within test borings B-1 and B-4 to depths of about 18 to 20 feet below existing ground surface and generally consisted of very dense silty sandstone materials.

Groundwater

Groundwater was encountered during our subsurface investigation to depths of about 17 and 18 feet below existing 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 can also cause fluctuations of local or shallow perched groundwater levels.

4.3 Photoionization Detector (PID) Screening

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

4.4 Infiltration Testing

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

Two percolation tests (designated as B-5 and B-6) were conducted and involved the drilling of the test boring utilizing a hollow-stem auger drill rig with an outside diameter of approximately 8 inches. The percolation test procedure by City of San Diego BMP Design Manual (2018) was used in our percolation tests.

The approximate percolation test boring locations are shown in the Test Boring Location Plan (Figure 1). A perforated 2-inch diameter pvc pipe was installed inside each of the test boring with gravel placed below and on the sides of the perforated pipe. The percolation tests involved presoaking the boring and filling the test holes with water, recording the drop in water surface with time, and refilling the holes with water. The results of the percolation test are presented on the following table.

The drop in water level over time is the percolation rate at the test location. The percolation rates were 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 infiltration rate.

Infiltration Rate = $\Delta H (60r) / \Delta t (r + 2H_{avg})$

Where: r is the radius of the test hole (in)
ΔH is the change in height over the time interval (in)
Δt is the time interval (min)
Havg is the average head height over the time interval

The design infiltration rate noted below has not been reduced to account for a factor safety (FS).

TABLE 1 – PERCOLATION TEST RESULTS

Test Hole	Test Depth[1] (feet)	Percolation Rate (in/hr)	Infiltration Rate (in/hr)	Soil Type
B-5	5.0	0.48	0.05	Clayey Sand
B-6	5.0	0.00	0.00	Sandy Clay

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.

Based on the results of the infiltration, it is our opinion that an on-site stormwater infiltration system is not suitable due to very low infiltration rates obtained during our testing.

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 on-site soils. The following are brief descriptions of our laboratory test results.

In Situ Moisture and Density

Tests were performed on select samples from the test borings to determine the subsoils dry density and natural moisture contents in accordance with Test Method ASTM 2216-05. The results of these tests are included in the Test Boring Logs enclosed in Appendix A.

Sieve Analysis

Sieve Analyses including Passing No. 200 sieve were performed on selected samples from various depths within Test Borings B-1 and B-5 to assist in soil classification and aid in the liquefaction analysis. These tests were performed in accordance with Test Method ASTM D 1140-00 (Reapproved 2006) and ASTC C 1369-96. The results of the sieve analysis are graphically presented as Figure 2 and passing no. 200 results are presented in Test Boring Logs.

Expansion

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Boring B-1 (1 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829-08a. The result of our expansion index (EI) test indicates that the near surface sample has a *very low* expansion potential (EI= 14).

Consolidation Test

Settlement prediction under anticipated load was made on the basis of one-dimensional consolidation test. These tests were performed in general accordance with Test Method ASTM D 2435 and ASTM 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 (1.25%) potential. The Consolidation test curve, Figure 3 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. The following table presents the results of our laboratory testing.

Parameter	**B-2 1 to 5 feet**
pH	7.48
Chloride	134 ppm
Sulfate	0.0162%
Resistivity	800 ohm-cm

The chloride content of the 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 soil has a Low exposure to chloride. The results of limited in-house testing of soil pH and resistivity were determined in accordance with California Test Method No. 643 and indicated that on-site soil is moderately alkaline with respect to pH and soil resistivity was found to possess a severe degree of corrosivity.

These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The test results on a near surface bulk sample from the site generally indicate that tested on-site soils have severe 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.

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample in accordance with California Test Method No. 417. Our laboratory test data indicated that near surface soils contain approximately 0.0162 percent of water soluble sulfates. Based on the 2016 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-05, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-05, a low exposure to sulfate corrosivity 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 CONCLUSIONS AND RECOMMENDATIONS

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

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

Impact of Site on Stability of Adjacent Properties

It is our opinion that the proposed grading and construction for the subject site will not affect adversely impact the stability of adjoining properties provided that grading and construction are performed in accordance with the recommendations provided herein and in accordance with local code guidelines.

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2016 California Building Code (CBC) and applicable local codes. Based on the results of our subsurface exploration, 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 2016 CBC, the Rose Canyon, Newport Inglewood, Coronado Bank and Elsinore faults are the closest known active faults and are located about 4.11, 4.11, 20.04 and 23.55 miles, respectively, to the site. The Newport Inglewood Fault would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 7.50.

The proposed structure should be designed in accordance with the current version of the 2016 California Building Code (CBC) and applicable local codes. Within the International Code Council's 2015 International Building Code (IBC), the five-percent damped design spectral response accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are used to determine the seismic design base shear. These parameters, which are a function of the site's seismicity and soil, are also used as parts of triggers for other code requirements. The following values are determined by using the USGS published U.S. Seismic Design Maps program based upon the 2016 CBC referenced ASCE 7 (with July 2013 errata).

CBC 2016, Earthquake Loads	
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.3.1(1) for 0.2 second)	1.160
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.3.1(2) for 1.0 second)	0.446
Site Coefficient, F_a (Table 1613.3.3 (1) short period)	1.036
Site Coefficient, F_v (Table 1613.3.3 (2) 1-second period)	1.554
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.202
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.693
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	0.801
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.462

Liquefaction

A site liquefaction evaluation consistent with the guidelines contained in CDMG Special Publication 117A along with a report by Southern California Earthquake Center (SCEC) has been performed as part of the current investigation. Our site-specific probabilistic seismic hazard analysis was derived using data published by the United States Geological Survey (USGS).

Based on 2016 CBC, Section 1803.5.12, Seismic Design Categories D through F, the peak ground acceleration shall be determined in accordance with Section 11.8.3 of ASCE 7. The predominant earthquake magnitude of 6.72 was obtained from the USGS Interactive Deaggregation web site using 2% probability of exceedance in 50 years. The mean peak ground acceleration for the site used in our liquefaction analysis was determined to be 0.482g.

Our liquefaction analysis was performed using the computer program Liquefypro (version 5) developed by Civil Tech Software. The program is based on the most recent publications of the NCEER Workshop and SP117 Implementation. Corrected SPT blow counts based upon hammer energy ratio, borehole diameter and sampling method were used in analysis calculations. Although groundwater was encountered at a depth of about 17 to 18 feet below existing ground surfaces during our drilling operations, groundwater of 10 feet was used in our liquefaction analysis. The liquefiable layers at the location of boring B-1 are presented graphically in Plate A1 of Appendix A. The computer output files are also included.

In order to estimate the amount of post-earthquake settlement, methods proposed by Tokimatsu and Seed (1987) were used for the settlement calculations. Based on our analysis and under the current site conditions, we estimate that the maximum total seismic-induced ground settlement at the site would be negligible (0.01 inch) and therefore, not significant to the proposed development.

6.2 Site Improvement Recommendations

The following recommendations for site development have been based upon the assumed floor elevation and foundation bearing grades and the conditions encountered at the test boring locations.

Site Clearing

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

All soils disturbed by the demolition of the existing improvements should be removed to expose a competent subgrade, as determined by the project geotechnical engineer. Debris resulting from the demolition and clearing operations should be legally exported from the site.

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

Building Area

Due to the presence of variable strength characteristics of the near surface soils and likely disturbance of site soils during clearing operations, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (5 feet minimum) be over-excavated to a depth of at least 2 feet below existing grade or planned grade and 1 foot below bottom of footings, whichever is greater. 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. Prior to placement of fill, the exposed surfaces approved for fill placement should be scarified to a depth of at least 12 inches, moisture conditioned 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 engineer should be present on site during grading operations to verify proper placement and adequate compaction of all fills.

Proofroll and Compact Subgrade

The subgrades within the new pavement area should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded dump truck to detect very loose/soft yielding soil which should be removed to a stable subgrade. Following proofrolling and completion of any necessary overexcavation, the subgrades should be scarified to a depth of at least 8 inches, moisture conditioned and recompacted to at least 90 percent of the Modified Proctor maximum dry density. In accordance with the enclosed Guide Specifications and in the event that new pavement is constructed within the site, the top 12 inches of the pavement subgrade soils should be compacted to at least 95 percent of the Modified Proctor maximum density, or, 5 percent higher than the underlying fill materials. Low areas and excavations may then be backfilled in lifts with suitable very 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 should be used as a minimum in developing the project specifications. The need may arise to recompact the floor slab and pavement subgrades immediately prior to construction due to the effects of weather and construction traffic on a previously prepared subgrade.

Reuse of On-site Soil

On-site material may be reused as structural compacted fill within the proposed building and pavement improvement area provided they are moisture conditioned and compacted as recommended, and do not contain oversized materials, 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 pavement support. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering controlled conditions.

Import Structural Fill

Any soil imported to the site (if required) for use as structural fill should consist of very low expansive soils (EI less than 21). Material designated for import should be submitted to the project geotechnical engineer no less than three working days prior to placement for evaluation.

In addition to expansion criteria, soils imported to the site should exhibit adequate characteristics for the recommended pavement support characteristics and soluble sulfate content.

Subgrade Protection

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

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

Fill Placement

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

All fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned and then compacted in accordance with recommendation herein and with the enclosed “Guide Structural Fill Specifications”. A representative of the geotechnical engineer should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

6.3 Construction Considerations

Construction Dewatering

As mentioned previously, groundwater was encountered at depths of about 17 and 18 feet below existing grade during our subsurface investigation. In the event that shallow perched water is encountered, filter sump pumps placed within pits in the bottoms of excavations are expected to be the most feasible method of construction dewatering.

Soil Excavation

Some slope stability problems may be encountered for shallow 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.

Due to the presence of dense to very dense on-site soils at shallow depths, some difficulty may be encountered during excavation with conventional equipment. The use of specialized excavation equipment may be necessary.

6.4 Foundation Recommendations

Vertical Load Capacity

Upon completion of the building pad preparation, the proposed structure may be supported by a shallow foundation system. The foundation system may consist of either independently constructed spread footings or monolithically constructed foundation and floor slab thereby using a turned-down slab construction technique. 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 16 and 24 inches, respectively, regardless of the calculated soil bearing pressure. The recommended allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Reinforcing

The recommended minimum quantity of longitudinal reinforcing for geotechnical considerations within continuous strip footing is four No. 5 bars (2 top and 2 bottom) continuous through column pads within the strip footings. The recommended quantity of longitudinal reinforcing pertains to a minimum 12-inch thick and a maximum 24-inch wide footing pad; additional reinforcing may be necessary if a thinner or wider footing pad is used to develop equivalent rigidity. Conventional reinforcing is considered suitable in isolated column pad footings. The final design of the foundations as well as determination of the actual quantity of steel reinforcing and the footing dimensions should be performed by the structural engineer.

Lateral Load Resistance

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

A coefficient of friction of 0.35 may be used with dead load forces for footings placed on competent native soil and/or newly placed compacted fill soil. An allowable passive earth pressure of 250 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against newly placed structural fill. The maximum recommended allowable passive pressure is 2,000 psf.

Bearing Material Criteria

Soil suitable to serve as the foundation bearing grade should exhibit at least a loose relative density (average N value of at least 10) for non-cohesive soils or possess a stiff consistency (average unconfined compressive strength of 1.50 tsf) for cohesive soils for the recommended 3,000 psf allowable soil bearing pressure. For design and construction estimating purposes, suitable bearing soils are expected to be encountered at nominal foundation depths following the recommended site preparation activities. However, field testing by the Geotechnical Engineer within the foundation bearing soils is recommended to document that the foundation support soils possess the minimum strength parameters noted above. If unsuitable bearing soils are encountered, they should be recompacted in-place, if feasible, or excavated to a suitable bearing soil subgrade and to a lateral extent as defined by Item No. 3 of the enclosed Guide Specifications, with the excavation backfilled with structural compacted fill to develop a uniform bearing grade.

Foundation Embedment

The California Building Code (CBC) requires a minimum 12-inch foundation embedment depth. However, it is recommended that exterior foundations extend at least 18 inches below the adjacent exterior grade for bearing capacity consideration. 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 Settlement

Post-construction total and differential static movement (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, for static conditions. The estimated differential movement is anticipated to result in an angular distortion of less than 0.002 inches per inch on the basis of a minimum clear span of 20 feet. The maximum estimated total and differential movement is considered within tolerable limits for the proposed structure provided it is considered in the structural design.

6.5 Floor Slab Recommendations

Subgrade

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

Design

The floor 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 minimum slab reinforcing for geotechnical considerations is recommended to consist of No. 3 rebars at 18 inches on center, each way. Based on the recommended reinforcing and the assumed live loading, 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. If desired, a Subgrade Modulus of 150 pci may be used for floor slab design.

The floor slab is recommended to be underlain by a 4 inch thick layer of granular material. A minimum 10-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture on future flooring materials. The vapor retarder is recommended to be in accordance with ASTM E 1745-11, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. The sheets of the vapor retarder material should be evaluated for holes and/or punctures prior to placement and the edges overlapped and taped. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of coarse sand (Sand Equivalent>30) approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of coarse sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Movements

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. The maximum total and differential movement is considered within tolerable limits for the proposed structure, provided that the structural design adequately considers this distortion.

6.6 Retaining Wall Recommendations (If Required)

It is possible that retaining walls may be needed for this site. The retaining wall(s) may be supported by conventional shallow spread footings designed for an allowable soil bearing pressure of 3,000 psf. A higher allowable soil bearing pressure may be possible, but that determination should be based on a review of the locations and details of the planned wall and foundation elevations.

Design of walls should incorporate an adequate factor-of-safety against both over-turning and sliding (FS=1.5). The overturning resultant should also fall within the center third (kern) of the retaining wall footing for stability, or the design must be re-evaluated with a reduced bearing area.

Static Lateral Earth Pressures

Retaining walls should be designed to resist the applicable lateral earth pressures. On-site soil materials may be used as backfill behind walls, provided they are confirmed to have very low expansive characteristic and allow for a drainage layer as discussed in subsequent paragraphs. For on-site soils and/or imported soils (EI less than 21) to be used as backfill materials, an active earth pressure of 35 pounds per cubic foot (equivalent fluid pressure) should be used assuming a level adjacent backfill and drained conditions. For walls to be restrained at the top, an at-rest pressure of 55 pcf should be used for design. All retaining walls should be supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings and vehicles in addition to the above recommended active earth pressure.

Crushed rock or clean sand and gravel exhibiting a sand equivalent of 30 or greater may also be used for retaining wall backfill. If these materials are used as backfill within the active zone, the retaining wall may be designed for an active earth pressure of 30 pounds per cubic foot (equivalent fluid pressure) and 45 pounds per cubic foot for at rest pressure.

Drainage and Damp-proofing

Retaining walls are recommended to be designed for drained earth pressures and therefore, adequate drainage should be provided behind the walls. This can be accomplished by installing subdrains at the base of the walls. Wall footing-drains should consist of a system of filter material and perforated pipe. The perforated pipe system should consist of 4-inch diameter, schedule 40, PVC pipe or equivalent, embedded in 1 cubic foot of Class II Permeable Material (CALTRANS Standard Specifications, latest edition) or equivalent per lineal foot of pipe. Alternatively, ¾-inch open graded gravel or crushed rock enveloped in Mirafi 140 geofabric or equivalent may be used instead of the

Class II Permeable Material. The pipe should be placed at the base of the wall, and then routed to a suitable area for discharge of accumulated water. Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Damp-proofing of walls below-grade is recommended especially where moisture control is required by an approved waterproofing compound or covered with similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Retaining wall backfill behind the drainage layers should consist of low expansive soils with an E.I. less than 21, as determined by ASTM D 4829-03 method. Wall backfill should not contain organic material, rubble, debris, and rocks or cemented fragments larger than 3 inches in greatest dimension. A 1 foot thick low-expansive cohesive layer or pavement should be placed at the surface to help prevent surface water intrusion. A geotextile or filter fabric should be placed between the granular drainage layers and adjacent soils (excavated face or compacted materials) to prevent fines from migrating into the drainage layers.

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

6.7 New Pavement

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

New Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of existing on-site soil that exhibit a very low to low expansion potential. An R-value of 20 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of Carlsbad 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	5	8	Section 26, Class 2 (R-value at least 78)

NOTES:
(a) Compaction to density between 95 and 100 percent of the 50-Blow Marshall Density
(b) The surface and binder course may be combined as a single layer placed in one lift if similar materials are utilized.

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

Portland Concrete Pavements

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

The maximum joint spacing within all of the Portland Cement Concrete pavements is recommended to be 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.

6.8 Recommended Construction Materials Testing Services

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

6.9 Basis of Report

This report is based on Giles' proposal, which is dated August 17, 2018 and is referenced by Giles' proposal number 2GEP-1808006. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

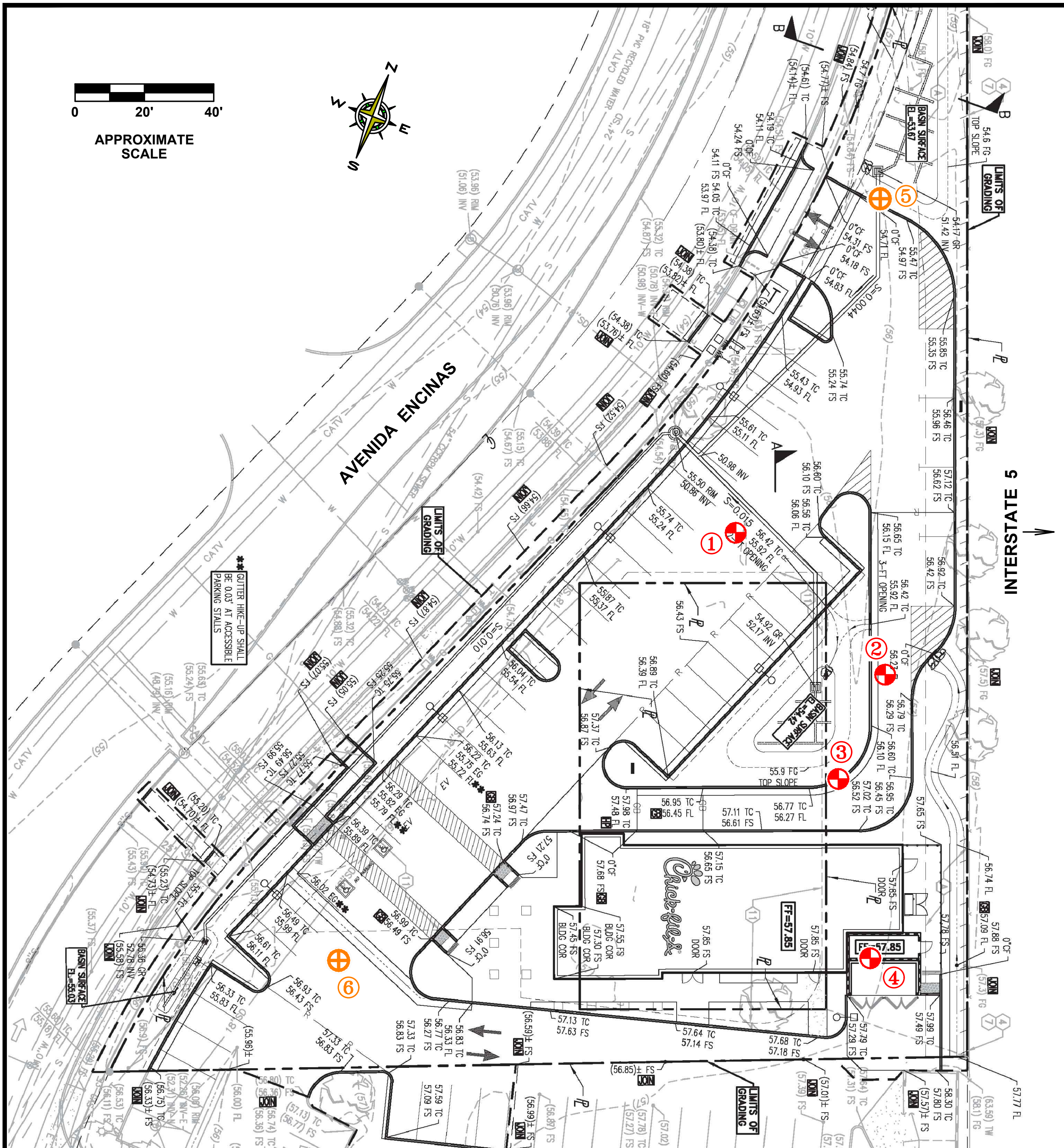
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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



LEGEND:

① GEOTECHNICAL TEST BORING

⑤ GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING

NOTES:

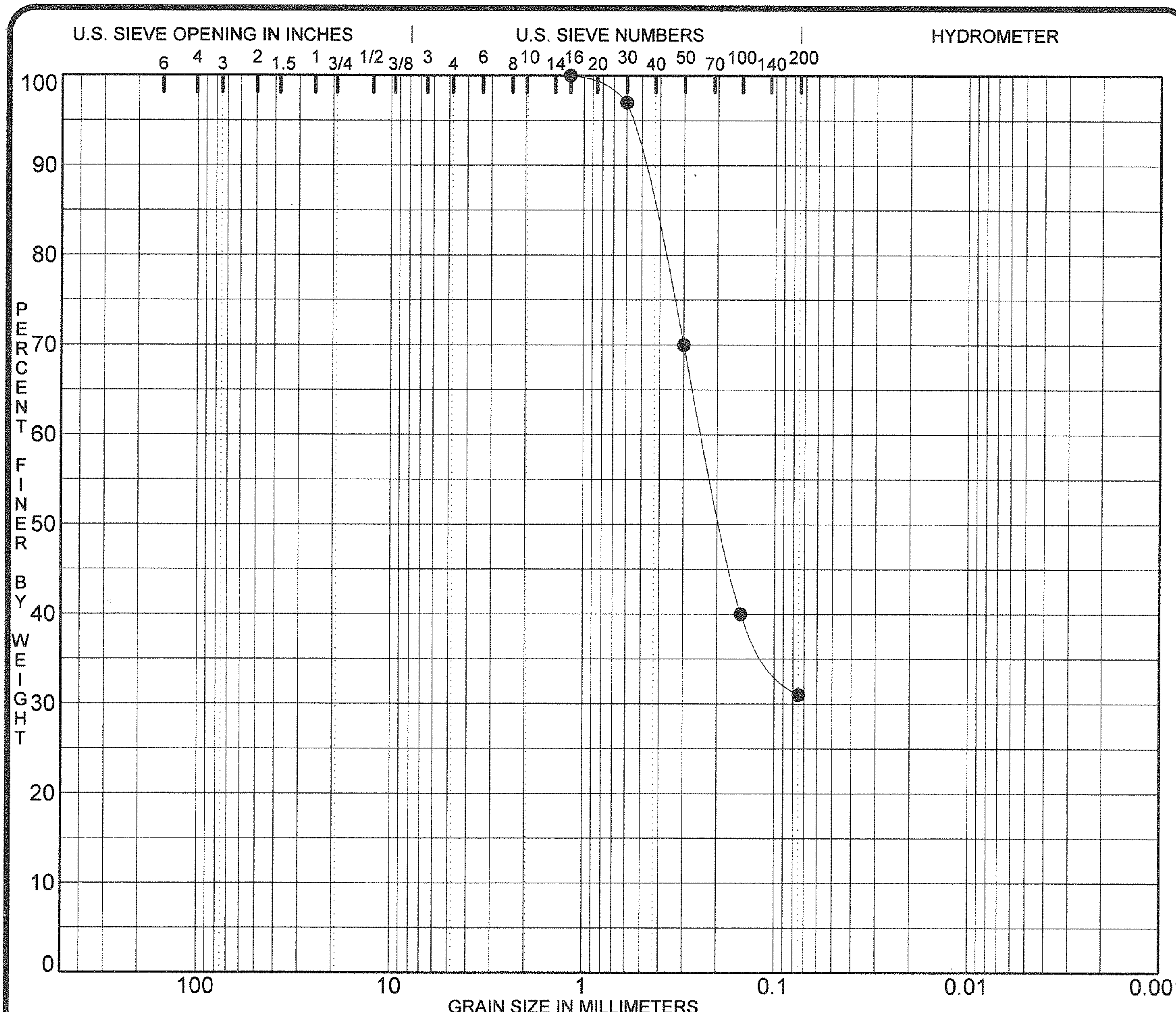
1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "CONCEPTUAL GRADING PLAN" (SHEET 2 of 5), DATED 3-1-19, PREPARED BY JOSEPH C. TRUXAW & ASSOCIATES, INC.

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

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 04306
I-5 AND PALOMAR FSU
5850 AVENIDA ENCINAS
CARLSBAD, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	Jdid	approx. 1"=40'	10-01-18	03-13-19
PROJECT NO.: 2G-1808005			CAD No. 2g1808005-blp2	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-5	3.5	**Clayey Sand to Silty Sand**						

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-5	3.5	**1.18**	**0.24**			**0.0**	**69.0**	**31.0**	

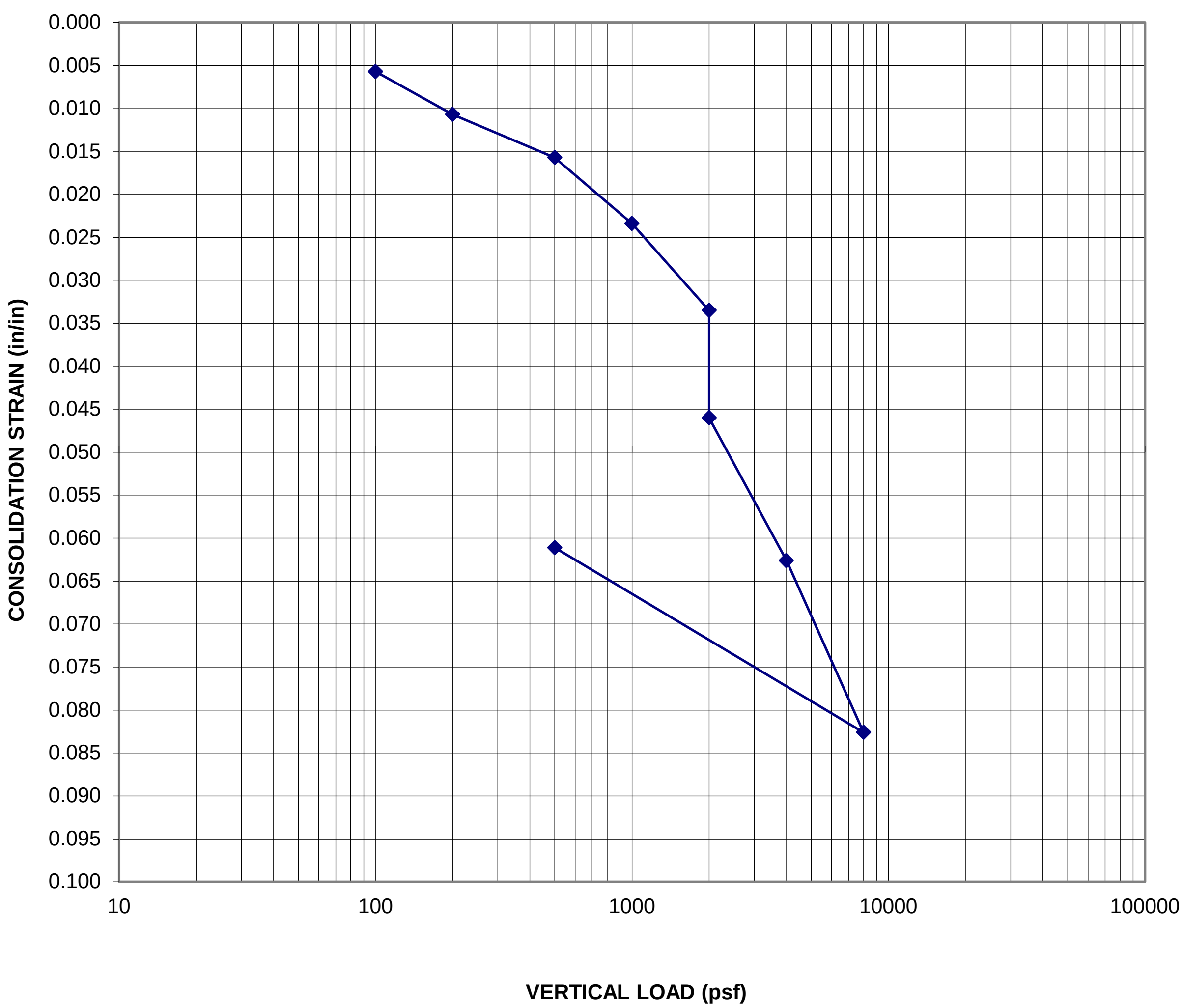
PROJECT **Proposed CFA #4306 - Carlsbad, CA**
FIGURE 2

JOB NO. **2G-1808005**
DATE **10/4/18**

GRADATION CURVES

Giles Engineering Associates, Inc

CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Clayey Sand		
Boring No.	B-3		
Sample No.	2-CS	Initial Moisture Content (%)	11.2
Depth (ft.)	3.0	Final Moisture Content (%)	15.5
Elevation		Natural Density (pcf)	123.4
Liquid Limit		Initial Dry Density (pcf)	110.9
Plastic Limit		Final Dry Density (pcf)	119.4
Specimen Diameter (in.)	2.42	Collapse @ 2000 psf	1.25%
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 2000 psf pressure

Project: CFA Carlsbad

Client: Chick-fil-A

Project No.: 2G-1808005

Figure No.: 3

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-

1965 NORTH MAIN STREET, ORANGE, CALIFORNIA

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

BORING NO. & LOCATION: B-1	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 56.5 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 2.5 inches of asphaltic concrete over 4.5 inches of aggregate base		55								
Brown Clayey fine Sand - Moist (Possible Fill)			1-SS	18				13	BDL	P_{200}=40%
Gray fine Sand, some Silt, some layers of Silty Sand - Moist (Native)	5	50	2-SS	20				13	BDL	P_{200}=23%
Light Brown Silty Sand to fine to medium Sand, trace Silt - Moist	10	45	3-SS	32				13	BDL	P_{200}=20%
	15	40	4-SS	51				14	BDL	
Light Yellowish Silty Sandstone - Moist (Old Paralic Deposits)	20	35	5-SS	50/3"				16	BDL	P_{200}=27%
	25	30	6-SS	50/6"				15	BDL	
	30	25	7-SS	50/5"				15	BDL	P_{200}=23%
	35		8-SS	50/4"				10	BDL	

Groundwater encountered at 18 feet
Boring Terminated at about 35.5 feet (EL. 21')

Water Observation Data	Remarks:
Water Encountered During Drilling: 18'	SS = Standard Penetration Test
Water Level At End of Drilling:	BDL - Below Detection Level
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: 57 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

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										
Light Brown Clayey Sand - Moist (Possible Fill)		55	1-SS	11				17	BDL	
Brown Clayey fine Sand - Moist (Native)	5		2-CS	48				14	BDL	Dd=124.9 pcf
Light Brown Silty Sand to fine Sand with Silt - Moist		50	3-CS	53				8	BDL	Dd=104.5 pcf
	10		4-CS	63				12	BDL	Dd=116.8 pcf

No groundwater encountered
Boring Terminated at about 10 feet (EL. 47')

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:	BDL - Below Detection Level
Water Level After Drilling:	
Cave Depth After Drilling:	

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

BORING NO. & LOCATION: B-3	TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 56.8 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

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										
Brown Clayey fine Sand - Moist (Possible Fill)		55	1-SS	9				20	BDL	
Brown to Light Brown Clayey fine Sand - Moist (Native)			2-CS	27				17	BDL	Dd=111.0 pcf
	5									
Yellowish Brown fine Sand to Silty fine Sand, some iron oxide staining - Moist		50	3-CS	40				8	BDL	Dd=112.3 pcf
			4-CS	46				13	BDL	Dd=104.7 pcf
	10									

No groundwater encountered
Boring Terminated at about 10 feet (EL. 46.8')

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:	BDL - Below Detection Level
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	GILES ENGINEERING ASSOCIATES, INC.
B-4		
SURFACE ELEVATION: 57.5 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Qu (tsf)	Qp (tsf)	Qs (tsf)	W (%)	PID	NOTES
Approximately 5 inches of asphaltic concrete										
Brown Clay fine Sand - Moist (Possible FIll)		55	1-SS	15				17	BDL	
Light Brown fine Sand, trace of Clay, some layers of Silty Sand - Moist (Native)	5		2-SS	25				10	BDL	
		50								
Brown fine Sand, trace to little Silt - Moist	10		3-SS	30				10	BDL	
		45								
	15		4-SS	35				16	BDL	
		40								
Yellowish Brown Silty Sandstone - Moist (Old Paralic Deposits)	20		5-SS	50/5"				11	BDL	

Groundwater encountered at 17 feet
Boring Terminated at about 21.5 feet (EL. 36')

Water Observation Data	Remarks:
Water Encountered During Drilling: 17'	SS = Standard Penetration Test
Water Level At End of Drilling:	BDL - Below Detection Level
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: 56.3 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

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										
Brown Clayey fine Sand to Silty fine Sand - Moist (Possible Fill to Native)		55.0	1-SS	17				14	BDL	
	2.5	52.5	2-SS	38				7	BDL	P_{200}=30%
	5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 51.3')

Water Observation Data	Remarks:
Water Encountered During Drilling: None	SS = Standard Penetration Test
Water Level At End of Drilling:	BDL - Below Detection Level
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-6		
SURFACE ELEVATION: 56.4 feet	PROPOSED CHICK-FIL-A RESTAURANT #4306	GILES ENGINEERING ASSOCIATES, INC.
COMPLETION DATE: 09/11/18	5850 AVENIDA ENCINAS CARLSBAD, CA	
FIELD REP: TREVOR SLAZAS	PROJECT NO: 2G-1808005	

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 5 inches of aggregate base										
Brown fine Sandy Clay - Moist (Possibble Fill to Native)		55.0	1-SS	5				25	BDL	
	2.5									
		52.5	2-SS	18				22	BDL	
	5.0									

No groundwater encountered
Boring Terminated at about 5 feet (EL. 51.4')

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

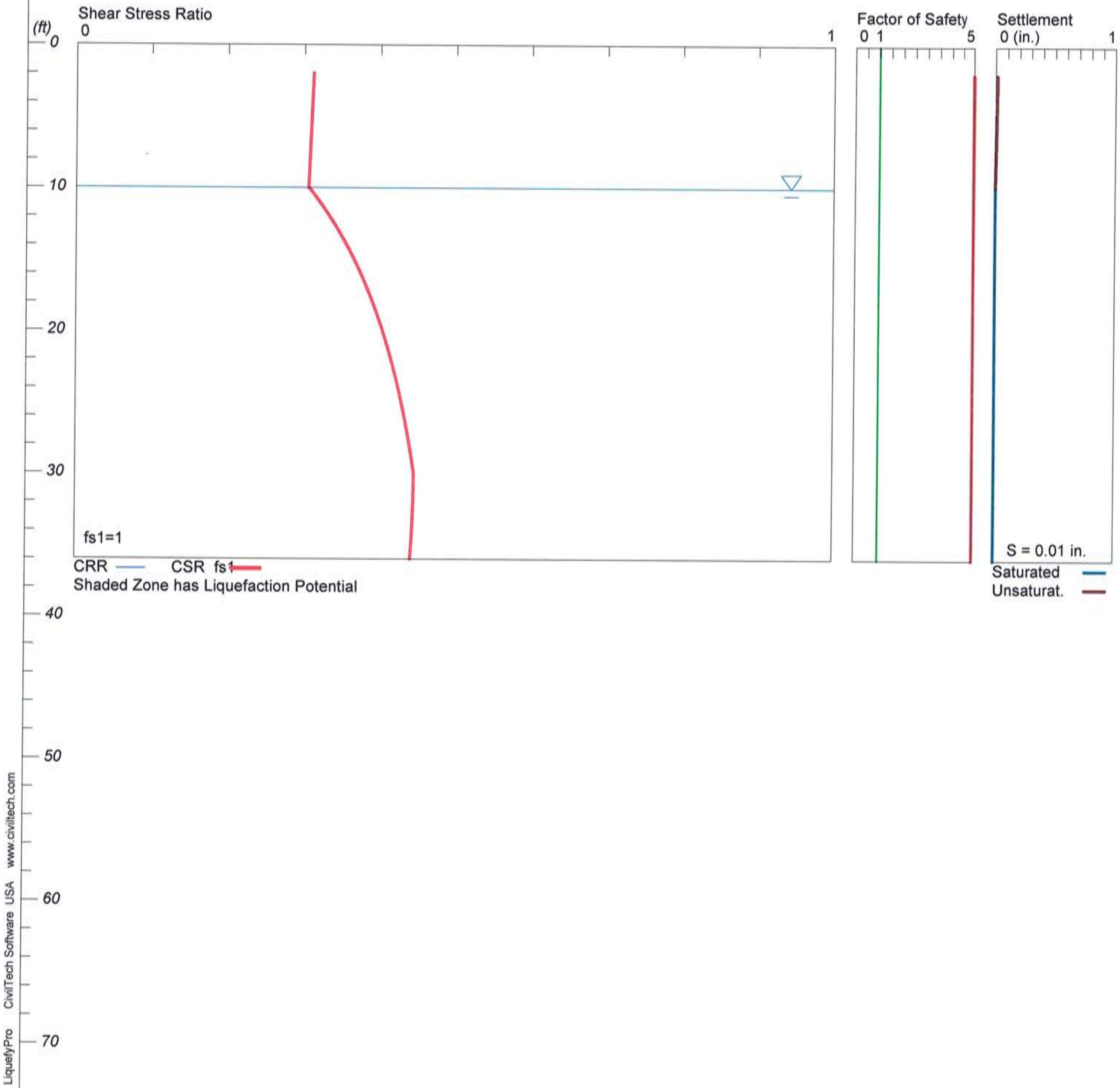
Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

LIQUEFACTION ANALYSIS

CFA #4306 - Carlsbad, CA

Hole No.=B-1 Water Depth=10 ft

Magnitude=6.72
Acceleration=0.482g



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LIQUEFACTION ANALYSIS SUMMARY

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Input File Name: UNTITLED
Title: CFA #4306 - Carlsbad, CA
Subtitle: 2G-1808005, 5850 Avenida Encinas

Surface Elev.=
Hole No.=B-1
Depth of Hole= 36.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 18.00 ft
Max. Acceleration= 0.48 g
Earthquake Magnitude= 6.72

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=36.00 ft
Water Table during Earthquake= 10.00 ft
Water Table during In-Situ Testing= 18.00 ft
Max. Acceleration=0.48 g
Earthquake Magnitude=6.72
No-Liquefiable Soils: CL, OL are Non-Liq. Soil

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

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
2.00	18.00	120.00	15.00
5.00	20.00	120.00	15.00
10.00	32.00	120.00	10.00
15.00	51.00	120.00	5.00
20.00	50.00	120.00	5.00
25.00	50.00	120.00	5.00
30.00	50.00	120.00	5.00
35.00	50.00	120.00	4.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.01 in.
Total Settlement of Saturated and Unsaturated Sands=0.01 in.
Differential Settlement=0.006 to 0.008 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
2.00	2.65	0.31	5.00	0.00	0.01	0.01
2.50	2.65	0.31	5.00	0.00	0.01	0.01
3.00	2.65	0.31	5.00	0.00	0.01	0.01
3.50	2.65	0.31	5.00	0.00	0.01	0.01
4.00	2.65	0.31	5.00	0.00	0.01	0.01
4.50	2.65	0.31	5.00	0.00	0.01	0.01
5.00	2.65	0.31	5.00	0.00	0.01	0.01
5.50	2.65	0.31	5.00	0.00	0.01	0.01
6.00	2.65	0.31	5.00	0.00	0.01	0.01
6.50	2.65	0.31	5.00	0.00	0.01	0.01
7.00	2.65	0.31	5.00	0.00	0.01	0.01
7.50	2.65	0.31	5.00	0.00	0.00	0.00
8.00	2.65	0.31	5.00	0.00	0.00	0.00
8.50	2.65	0.31	5.00	0.00	0.00	0.00
9.00	2.65	0.31	5.00	0.00	0.00	0.00
9.50	2.65	0.31	5.00	0.00	0.00	0.00
10.00	2.65	0.31	5.00	0.00	0.00	0.00
10.50	2.65	0.31	5.00	0.00	0.00	0.00
11.00	2.65	0.32	5.00	0.00	0.00	0.00
11.50	2.65	0.33	5.00	0.00	0.00	0.00
12.00	2.65	0.33	5.00	0.00	0.00	0.00
12.50	2.65	0.34	5.00	0.00	0.00	0.00
13.00	2.65	0.35	5.00	0.00	0.00	0.00
13.50	2.65	0.35	5.00	0.00	0.00	0.00
14.00	2.65	0.36	5.00	0.00	0.00	0.00
14.50	2.65	0.36	5.00	0.00	0.00	0.00
15.00	2.65	0.37	5.00	0.00	0.00	0.00
15.50	2.65	0.37	5.00	0.00	0.00	0.00
16.00	2.65	0.37	5.00	0.00	0.00	0.00
16.50	2.65	0.38	5.00	0.00	0.00	0.00
17.00	2.65	0.38	5.00	0.00	0.00	0.00
17.50	2.65	0.39	5.00	0.00	0.00	0.00
18.00	2.65	0.39	5.00	0.00	0.00	0.00
18.50	2.65	0.39	5.00	0.00	0.00	0.00
19.00	2.65	0.40	5.00	0.00	0.00	0.00
19.50	2.65	0.40	5.00	0.00	0.00	0.00
20.00	2.65	0.40	5.00	0.00	0.00	0.00
20.50	2.65	0.41	5.00	0.00	0.00	0.00
21.00	2.65	0.41	5.00	0.00	0.00	0.00
21.50	2.65	0.41	5.00	0.00	0.00	0.00
22.00	2.65	0.41	5.00	0.00	0.00	0.00
22.50	2.65	0.42	5.00	0.00	0.00	0.00
23.00	2.65	0.42	5.00	0.00	0.00	0.00
23.50	2.65	0.42	5.00	0.00	0.00	0.00
24.00	2.65	0.42	5.00	0.00	0.00	0.00
24.50	2.65	0.43	5.00	0.00	0.00	0.00
25.00	2.65	0.43	5.00	0.00	0.00	0.00
25.50	2.65	0.43	5.00	0.00	0.00	0.00
26.00	2.65	0.43	5.00	0.00	0.00	0.00
26.50	2.65	0.43	5.00	0.00	0.00	0.00
27.00	2.65	0.44	5.00	0.00	0.00	0.00
27.50	2.65	0.44	5.00	0.00	0.00	0.00
28.00	2.65	0.44	5.00	0.00	0.00	0.00
28.50	2.65	0.44	5.00	0.00	0.00	0.00

29.00	2.65	0.44	5.00	0.00	0.00	0.00
29.50	2.65	0.44	5.00	0.00	0.00	0.00
30.00	2.65	0.45	5.00	0.00	0.00	0.00
30.50	2.65	0.45	5.00	0.00	0.00	0.00
31.00	2.65	0.45	5.00	0.00	0.00	0.00
31.50	2.65	0.45	5.00	0.00	0.00	0.00
32.00	2.65	0.45	5.00	0.00	0.00	0.00
32.50	2.65	0.45	5.00	0.00	0.00	0.00
33.00	2.65	0.44	5.00	0.00	0.00	0.00
33.50	2.65	0.44	5.00	0.00	0.00	0.00
34.00	2.65	0.44	5.00	0.00	0.00	0.00
34.50	2.65	0.44	5.00	0.00	0.00	0.00
35.00	2.65	0.44	5.00	0.00	0.00	0.00
35.50	2.65	0.44	5.00	0.00	0.00	0.00
36.00	2.65	0.44	5.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

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

1 atm (atmosphere) = 1 tsf (ton/ft2)

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

APPENDIX B

FIELD PROCEDURES

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

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

GENERAL FIELD PROCEDURES

Test Boring Elevations

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

Test Boring Locations

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

Water Level Measurement

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

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

Borehole Backfilling Procedures

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

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

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

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

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

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

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

Bulk Sample (BS)

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

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

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

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

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

Sampling and Testing Procedures

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

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

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

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

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

Moisture Content (w) (ASTM D 2216)

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

Unconfined Compressive Strength (qu) (ASTM D 2166)

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

Calibrated Penetrometer Resistance (qp)

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

Vane-Shear Strength (qs)

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

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

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

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

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

Consolidation Test (ASTM D 2435)

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

Classification of Samples

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

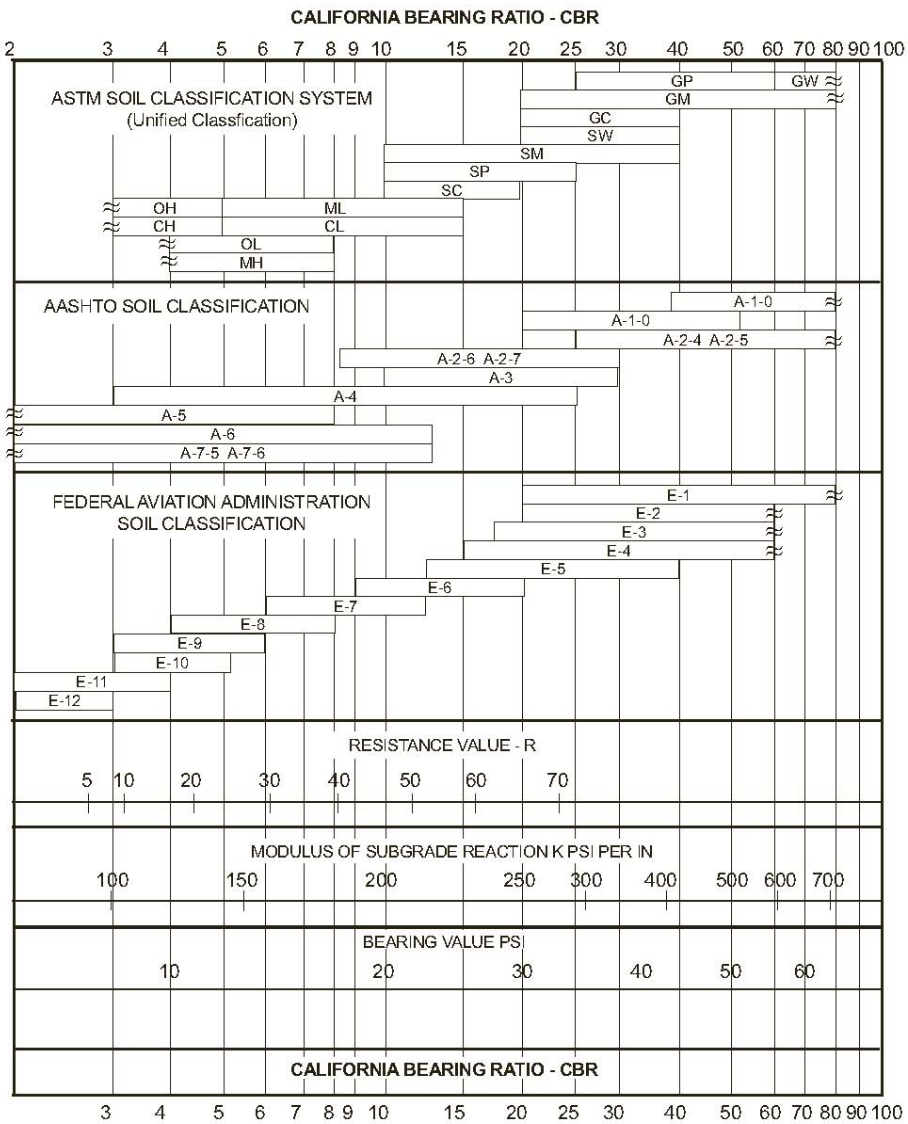
Laboratory Testing

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

California Bearing Ratio (CBR) Test ASTM D-1833

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

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



APPENDIX D

GENERAL INFORMATION

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

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

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

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

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

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

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

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

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

GENERAL COMMENTS

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

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

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

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

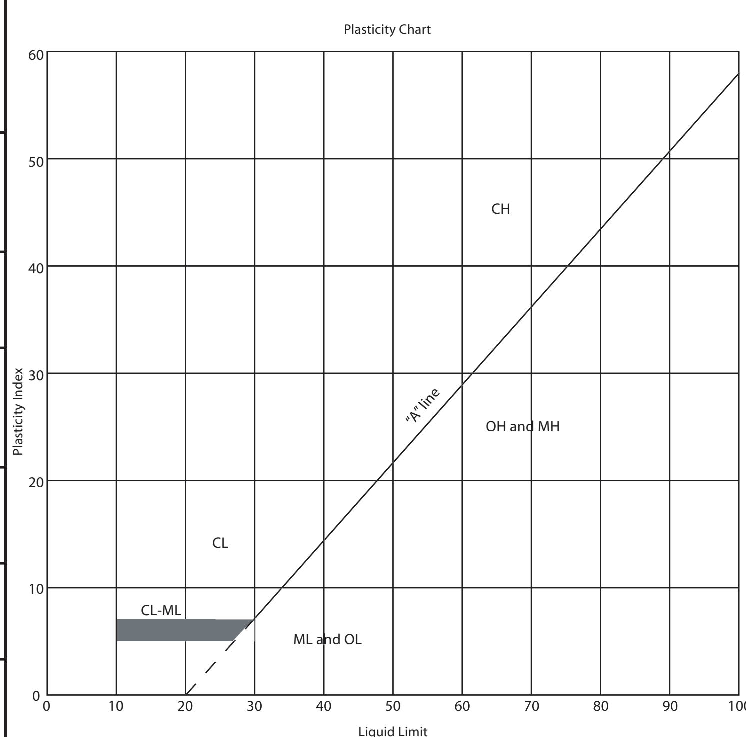
The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

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

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

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information About Your
Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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