Appendix D

Geotechnical Report

🟢 **GREEN**

Geotechnical Engineering Exploration and Analysis
DRAFT

Proposed Chick-fil-A Restaurant #5420
Hwy 111 and Dune Palms Road FSU
NEC Highway 111 and Dune Palms Road
La Quinta, California

Prepared for:

Chick-fil-A, Inc.
Irvine, California

Prepared by:

Giles Engineering Associates, Inc.

January 18, 2023
Project No. 2G-2210007

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

- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

January 18, 2023

Chick-fil-A, Inc.
105 Progress, Suite 100
Irvine, California 92618

Attention: Ms. Brenda Porrazzo
Sr. Specialist, Restaurant Development

Subject: Geotechnical Engineering Exploration and Analysis - Draft
Proposed Chick-fil-A Restaurant #5420
Hwy 111 and Dune Palms Road FSU
NEC Highway 111 and Dune Palms Road
La Quinta, California
Project No. 2G-2210007

Dear Ms. Porrazzo:

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

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

Respectfully submitted,

GILES ENGINEERING ASSOCIATES, INC.

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LA QUINTA, CALIFORNIA
PROJECT NO. 2G-2210007

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS - DRAFT

PROPOSED CHICK-FIL-A RESTAURANT #5420
HWY 111 AND DUNE PALMS ROAD FSU
NEC HIGHWAY 111 AND DUNE PALMS ROAD
LA QUINTA, CALIFORNIA
PROJECT NO. 2G-2210007

EXECUTIVE SUMMARY OUTLINE

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

Subsurface Conditions

- Site Class designation D is recommended for seismic design considerations.
- Our review of the *Geologic Map of the Palm Desert & Coachella 15-Min. Quadrangles (Dibblee, Jr., 2008)* indicates that the subject site is underlain by alluvial sand and clay of valley areas (Qa). Additionally, according to this plan, the subject site is located near to wind-laid dune sand (Qs) and alluvial sand and gravel of Whitewater River (Qg) deposits.
- Fill and possible soils were encountered in the test borings at depths of approximately 3½ feet below grade. Fill and native soils have very similar characteristics, and it was difficult to determine the exact contact. Fill consisted of very loose to loose, dry, silty sand, fine grained and variable amounts of mica.
- Native soils were encountered within our test borings, which were generally loose to medium dense in relative density silty sand and poorly-graded sand with silt, dry to moist, fine grained, with various amount of mica and medium stiff to stiff in comparative density sandy silt, fine sand, moist, with some mica.
- Groundwater was not encountered in the Test Borings drilled to a maximum depth of 16.5 feet bgs.
- Due to the sandy nature, the on-site soils generally possess a very low expansion potential.
- The results from the minimum resistivity test generally indicate that the tested soils have a **moderate corrosive potential** when in contact with ferrous materials.

Site Development

- The proposed site development will include the construction of a new Chick-fil-A single-story building and site improvements that will include drive-thru lane with canopies, parking stalls, menu board signs, new trash enclosure, concrete walkways, and planter areas.
- New Building: Due to the variable and low strength characteristics of the near surface onsite soils and the presence of fill and possible fill materials, and to develop uniformity of support, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (5 feet minimum) be cut and filled as necessary to develop a minimum 2-foot structural fill layer below the foundations and floor slab. For the planned subgrade, the existing soils should be proofrolled to remove any unstable materials and the surface compacted to an in-place density of at least 90% of its maximum dry density per ASTM D-1557. The existing soils are considered suitable for foundation and floor support with the recommended 2-foot structural fill layer and for pavement support with recommended proofroll and geotechnical inspection/testing. The soils exposed after cutting to the structural fill subgrade 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 6 to 8 inches, moisture conditioned above optimum moisture content and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557).

Building Foundation

- The proposed structure may be supported by a shallow spread footing foundation system or turned-down slabs designed for a maximum, net allowable soil bearing pressure of 3,000 pounds per square foot (psf) underlain by a minimum 2-foot structural compacted fill layer.
- Foundation reinforcement should be determined by the structural engineer.

Building Floor Slab

- It is recommended that an on-grade slab be a minimum 4-inch-thick slab-on-grade or turned-down slab, underlain by a minimum 4-inch-thick granular base supported on a properly prepared subgrade consisting of a minimum 1-foot structural fill layer.
- A minimum 15-mil vapor barrier is recommended to be directly below the floor slab or base course where required to protect moisture sensitive floor coverings.

New Pavement

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

Construction Considerations

- The near surface soils consist mostly of very loose to loose, silty sand, fine grained, and may be unstable in steep, unbraced excavations.

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.

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. (“Giles”) conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report. The scope of work performed for this report was consistent with the scope of work outlined within Proposal No. 2GEP-2210025.

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

Giles conducted a Phase I Environmental Site Assessment (ESA) for the subject site. The results of that assessment are provided under separate cover (2E-2210011).

2.0 SITES AND PROJECT DESCRIPTION

2.1 Site Description

A new Chick-fil-A restaurant is to be constructed at the northeastern quadrant of Highway 111 and Dune Palms Road, in the City of La Quinta, Riverside County, California. The site is a vacant lot covered with soil and some bushes. The site is bordered on the north by a vacant lot, on the east by a vacant lot and some commercial developments, on the south by Highway 111, and on the west by Dune Palms Road. The property is situated at approximately latitude 33.7078° North and longitude -116.2765° West.

Based on Google Earth Pro, the site has a gently undulated slope, sloping down from Dune Palms Road (approximate El. 58) toward the eastern area, with the lowest elevation at the northeastern area (approximate El. 53). Highway 111 seems to follow the same sloping surface condition along the southern perimeter of the property.

2.2 Proposed Project Description

The proposed site development will include the construction of a new Chick-fil-A single-story building and site improvements that will include drive-thru lane with canopies, new parking stalls, menu board signs, a trash enclosure, concrete walkways, outside dining area, and planter areas. Although detailed building plans are not yet ready for our review, the new building will be a single-story wood-frame structure, 4,850 square feet gross area, with no basement or underground levels. We were not provided with specific loading information for this project at the time of this report; however, based on previous experience with similar projects, we expect the maximum combined dead and live loads supported by the bearing walls and columns will be 2 to 3 kips per lineal foot (klf) and 40 to 50 kips, respectively. The live load supported by the floor slab is expected to be a maximum of 100 pounds per square foot (psf).

Preliminary project information did not indicate the planned finished floor elevation for the proposed building. However, it is anticipated that the finish floor elevation of the new building will relatively match the existing grade, with a finish floor elevation of approximately El. 56. Therefore, site grading is anticipated to include only minor cutting or filling in order to establish the necessary site grade to accommodate the assumed 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 equivalent 18-kip single axle loads and 1,500 automobiles within the main drive lanes and only automobiles of a lesser intensity within the parking stalls. Pavement designs are based on a 20-year design period. Therefore, the parking lot pavement sections have been designed on the basis of a Traffic Index (TI) of 4.0 for the automobile traffic parking stalls (light duty) and a TI of 5.0 for drive lane areas (medium duty).

2.3 Background Information

The subject property is currently a vacant lot that has not been developed. A roadway was present in 1932 where the existing Highway 111 is now. Adjacent properties appeared to have been developed for use as agricultural and commercial land.

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of seven (7) test borings (B-1 through B-7) to depths of approximately 5 to 21.5 feet below existing ground surface utilizing a truck rig with hollow-stem auger drilling equipment. 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 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.

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

Our subsurface exploration planned to include the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes in accordance with ASTM D 3550, Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils. However, due to very loose to loose and very granular material encountered during field exploration, it was not possible to collect such samples.

3.2 Subsurface Conditions

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

Site Geology

Our review of the *Geologic Map of the Palm Desert & Coachella 15-Min. Quadrangles (Dibblee, Jr., 2008)* indicates that the subject site is underlain by alluvial sand and clay of valley areas (Qa). Additionally, according to this plan, the subject site is located near to wind-laid dune sand (Qs) and alluvial sand and gravel of Whitewater River (Qg) deposits.

Soil

Fill and possible soils were encountered in the test borings to depths of approximately 3½ feet below grade. Fill and native soils have very similar characteristics, and it was difficult to determine the exact geologic contact. Fill consisted of very loose to loose, dry, silty sand, fine grained and variable amounts of mica.

Beneath the fill, native soils were encountered within our test borings, which were generally loose to medium dense in relative density silty sand and poorly-graded sand with silt, dry to moist, fine grained, with various amount of mica and medium stiff to stiff in comparative density sandy silt, fine sand, moist, with some mica.

Groundwater

Groundwater was not encountered in any of the test borings drilled to a maximum depth of 21.5 feet bgs. Based on a review of City of La Quinta General Plan Seismic Hazards Map, Exhibit IV-3 (2011), the groundwater elevation is approximately -10 feet, which is approximately more than 60 feet in depth from the existing ground surface.

Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site could also cause fluctuations of local or shallow perched groundwater levels.

3.3 Percolation Testing

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

The percolation testing consisted of drilling an 8-inch-diameter hole using a hollow-stem auger, installing a 2-inch-diameter slotted pvc casing with a solid end cap and then surrounding the casing with a granular filter pack. The test holes (B-6 and B-7) were then pre-soaked to a minimum depth of 1 foot above the bottom of the boring. After pre-soaking, test water was added to the casing and refilled after each consecutive percolation test reading. The drop in water level over time is the percolation rate at the test location. The percolation test procedure outlined in the Riverside County Department of Environmental Health was used as a guide in our testing. A summary of the result of the percolation test is provided in Table below.

The drop in water level over time is the pre-adjusted percolation rate at the test location. The percolation rate was reduced to account for the discharge of water from the sides and bottom of the boring. The Porchet Method, noted below, was used to calculate for the design infiltration rate.

Tested 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)
H_{avg} is the average head height over the time interval

The results obtained from our percolation testing are summarized below. The infiltration rates noted below have not been reduced to account for a factor of safety.

Test Number	Test Depth (feet)	Design Infiltration Rate (in/hr)	Soil Type
B-6	0 - 5	56.8	(SM) – Silty Sand
B-7	5 - 10	18.0	(ML) – Sandy Silt

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.

3.4 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. PID responses between 0.5 and 5.0 (BDL - below the detectable limit of the instrument) instrument units were observed in the test borings drilled at the site, and considered insignificant. PID screening results are included on the soil Test Boring Logs.

4.0 LABORATORY TESTING

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

In Situ Moisture and Density

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

Expansive Potential

The expansive potential of the near surface soils was not evaluated for this site due to the very granular non-expansive material of the upper soils. Therefore, the expansion index (EI) should be considered as a very low to none expansion potential (EI=0).

Sieve Analysis

Sieve Analyses (Passing No. 200 Sieve) were performed on selected samples to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 1140. The results of the Passing No. 200 Sieve tests are presented in Test Boring Logs in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

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

Parameter	**B-1 through B-7** **1 to 5 feet**
pH	7.9
Chloride	123 ppm
Sulfate	0.0041%
Resistivity	2,000 ohm-cm

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

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

A representative sample of the near surface soils which may contact shallow buried utilities and structural concrete was performed to determine the concentrations present of water-soluble sulfate which could result in chemical attack of cement. Our laboratory test data indicated that **near surface soils contain approximately 0.0041 percent of water-soluble sulfates**. Based on Section 1904.1 of the California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318 a negligible exposure to sulfate can be expected for concrete placed in contact with the tested on-site soils. **No special sulfate resistant cement is considered necessary for concrete** which will be in contact with the tested on-site soils.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 Liquefaction

According to the City of La Quinta General Plan Seismic Hazards Map, Exhibit IV-3 (2011), the site is located outside the liquefaction hazard areas. Also, depth to groundwater at this site is deeper than 60 feet below grade. No groundwater was encountered during our field exploration to a maximum depth of 21.5 feet below ground surface (bgs). Therefore, a liquefaction analysis for this site was not necessary.

5.2 Active Fault Zones

Based on the City of La Quinta General Plan Seismic Hazards Map, Exhibit IV-2 (2011), the site is not located within any active faults zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

Effect of Proposed Grading and Construction on Adjacent Property

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

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

The site is not located within any active 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 *California Building Code (CBC)* and applicable local codes. In accordance with *ASCE 7*, Chapter 20, a Site Classification D is recommended for this site based upon the mapped geological features of the site also verified by test borings.

According to the maps of known active fault near-source zones to be used with the CBC, the S. San Adreas fault and Eureka Peak fault are the closest known active faults and located about 5.2 and 17.0 miles from the site, respectively. The San Andreas fault would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 8.18 (Hanks).

The proposed structure should be designed in accordance with the current version of the *California Building Code (CBC)*, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures ASCE 7*, and applicable local codes. The following values are determined by using the SEAOC/OSHPD Seismic Design Map Tool based upon the *CBC 2022* and *ASCE 7-16*.

CBC 2022, Earthquake Loads	
Site Class Definition (Table 20.3-1 from ASCE 7-16)	D
Mapped Spectral Response Acceleration Parameter, S_s (for 0.2 second)	1.654g
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.677g
Site Coefficient, F_a short period	1.00
Site Coefficient, F_v 1-second period	1.70
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS}	1.654g
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1}	1.151g
Design Spectral Response Acceleration Parameter, S_{DS}	1.102g
Design Spectral Response Acceleration Parameter, S_{D1}	0.767g
MCEG Peak Ground Acceleration adjusted for site class effects, PGA_M	0.788g

According to Section 11.4.8 of ASCE 7-16 for structural engineering considerations, a ground motion hazard analysis is required and should be performed in accordance with Section 21.2 for structures on Site Class D with S_1 greater than or equal to 0.2. However, as an exception to performing the ground motion hazard analysis, the value of the Seismic Response Coefficient (Cs) must be determined by Equation (12.8-2) for values of the fundamental period of the building (T) ≤ 1.5Ts, and taken as 1.5 times the value computed in accordance with either Equation (12.8-3) for T_L ≥ 1.5Ts, or Equation (12.8-4) for T > T_L.

6.2 Site Development Recommendations

The recommendations for site development as subsequently described are based upon the conditions encountered at the test boring locations and the results of our laboratory testing.

Site Clearing and Preparation

Clearing and demolition operations should include the removal of all landscape vegetation and any existing structural features, within the area of the proposed new building and site improvements. All soils disturbed by grading operations should be removed and/or compacted to provide a competent subgrade, as determined by the project geotechnical engineer.

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

Existing Utilities

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

Building Area

Due to the variable and low strength characteristics of the near surface onsite soils and the presence of fill and possible fill materials, and to develop uniformity of support, it is recommended that the soils within the proposed new building area and an appropriate distance beyond (5 feet minimum) be cut and filled as necessary to develop a minimum 2-foot structural fill layer below the foundations and floor slab. For the planned subgrade, the existing soils should be proofrolled to remove any unstable materials and the surface compacted to an in-place density of at least 90% of its maximum dry density per ASTM D-1557. The existing soils are considered suitable for foundation and floor support with the recommended 2-foot structural fill layer and for pavement support with recommended proofroll and geotechnical inspection/testing. The soils exposed after cutting to the structural fill subgrade 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 6 to 8 inches, moisture conditioned to above the soil's optimum moisture content, and then recompacted to at least 90% of the maximum dry density as determined by Modified Proctor (ASTM D 1557).

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

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

Proofroll and Compact Subgrade

Following site clearing, removal or re-compaction of disturbed soils and lowering of site grades where necessary for the 2-foot structural fill layer in the building area, the subgrades within the proposed building, pavement and drive thru areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade, or stabilized in place. Depending on examination by the geotechnical engineer, some over-excavation as previously indicated may be required and should be budgeted accordingly. Any unsuitable materials discovered should be removed and backfilled with structural fill. Following proofrolling and completion of any necessary over-excavation, the subgrades in the building, parking lot and drive thru areas should be scarified to a depth of 6 to 8 inches, moisture conditioned above the soil's optimum moisture content, and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable low-expansive structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

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

Reuse of On-site Soil

On-site material may be reused as structural compacted fill (if needed) within any new construction area provided they do not contain oversized materials and significant quantities of organic matter or other deleterious materials. All subgrade soil compaction as well as the selection, placement and compaction of new fill soils should be performed in accordance with the project specifications under engineering-controlled conditions.

Subgrade Protection

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

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

Fill Placement

All fill should be placed in 8-inch-thick maximum loose lift, moisture conditioned above the soil's optimum moisture content and then compacted to at least 90 percent, or 95 percent in the upper 12 inches of pavement subgrade) of the Modified Proctor maximum density. A representative of the project geotechnical consultant should be present on-site during grading operations to document proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Import Structural Fill

Any soils imported to the site (if required) for use as structural fill should consist of very low (EI less than 20) expansive soils. Materials designated for import should be submitted to the project geotechnical engineer no less than three working days for evaluation. In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure, soluble sulfate content and corrosivity and pavement support characteristics.

6.3 Construction Considerations

Construction Dewatering

Groundwater was not encountered during our subsurface exploration to the maximum depth explored (16.5 feet). However, the site may be susceptible to a shallower perched water table due to seasonal precipitation and runoff characteristics of the site. Conventional filtered sump pumps placed in excavations are expected to be suitable for dewatering should any excess water conditions be observed.

Soil Excavation

Some localized slope stability problems may be encountered in steep, unbraced excavations considering the nature of the subsoils. All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Due to anticipated on-site very loose to loose and granular material, shallow excavations, up to 4 feet in vertical height, may be adequately sloped for bank stability, where sufficient space is available, temporarily unsurcharged embankments could be sloped back at a 2:1 (h:v) slope gradient. Deeper excavations or excavations where adequate back sloping cannot be performed may require some form of external support such as shoring or bracing.

6.4 Foundation Recommendations

Vertical Load Capacity – Shallow Foundation

Upon completion of the recommended building pad preparation, it is our opinion the proposed structure may be supported by a shallow foundation system. Foundations underlain by a minimum 2-foot structural fill layer may be designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf). Minimum foundation widths for walls and columns should be 18 and 24 inches, respectively, for bearing considerations, regardless of actual soil pressure. The maximum bearing value applies to combined dead and sustained live loads. This allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

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 0.40 may be used with dead load forces for footings placed on newly placed compacted fill soil. An allowable passive earth pressure of 300 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 3,000 psf.

Foundation Embedment

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

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than 1 and ½ inch, respectively. The estimated differential movement is anticipated to result in an angular distortion of about 0.002 inches per inch on the basis of a minimum clear span of 20 feet.

Drilled Pier Design

It is our understanding that construction of canopies will be part of this development. Deep footings for canopies are assumed to typically be 3 feet in diameter, about 5 feet in length, reinforced concrete caissons. For this foundation system embedded into fill or native material encountered within our borings, the axial (downward) skin friction (side resistance) resistance was determined to be 100 psf based on the strength of the soils obtained from our field data and laboratory test results.

As an alternative, canopies may be supported on a pile foundation type based on an allowable bearing capacity of 1,000 psf for bearing into native soil, or 3,000 psf if underlain by 2 feet of structural fill. We recommend a minimum pile spacing of 3 pier diameters with no reduction in axial capacity for group effects. The minimum recommended pile length is 5 feet.

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

For upward pile resistance may be taken as equal to one-half the downward capacities, an average allowable side resistance of 50 psf may be used for the piers.

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

Pier Settlement Estimates and Considerations

Post-construction total and differential settlements of a pier foundation system designed in accordance with this report are estimated to be less than ¾ and ⅓ inch, respectively. The angular distortion will be less than 0.0014 inch per inch across the planned span of 20 feet. The estimated settlements are considered within tolerable limits for the proposed structure provided they are appropriately considered in the structural design. Estimated settlements are based on the assumption that foundation support soil will be tested and approved by a geotechnical engineer as well as the drilled pier construction will be observed by a geotechnical engineer during construction.

General Drilled-Pier Construction Recommendations

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

Pier excavations are expected to be cased due to the sandy soil. An uncased pier excavation should not be approached, as it could rapidly cave.

It is recommended that a geotechnical engineer monitor the drilling operations to confirm that proper construction techniques are used, and soil encountered within our borings is similar to soil encountered within the boreholes. Strict safety precautions must be implemented and followed when near open excavations, such as pier excavations. Concrete is recommended to be placed in accordance with "state-of-the-practice" procedures under engineering-controlled conditions as noted below. Drilled pier construction should be done in accordance with local codes, and other pertinent requirements.

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

Drilled Pier Lateral Loads

Resistance to lateral loads will be provided by the drilled piers. Active, At-Rest, and Passive Resistance (Equivalent Fluid Pressures) of 40 pcf, 60 pcf, and 360 pcf may be used for soil parameters, respectively. Reduction factors may be needed for group action for lateral capacities, dependent on the configuration of pier groups and the direction of applied lateral loads. The maximum recommended allowable passive pressure is 3,000 psf.

Reinforcing

The determination of the actual quantity of steel reinforcing and dimensions should be performed by the project structural engineer.

Bearing Material Criteria

Soil suitable to serve as the structural fill subgrade should exhibit at least a loose relative density (average N value of at least 8) for non-cohesive soils, and an unconfined compressive strength of 1.5 tsf for cohesive soils, for the recommended 3,000 psf allowable soil bearing pressure. For design and construction estimating purposes, suitable structural fill subgrade soils are expected to be encountered at the recommended 2-foot structural fill depths following the recommended site preparation activities. However, field testing by the Geotechnical Engineer within the structural fill subgrade and the structural fill 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.

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 including a minimum 2-foot structural fill layer. Foundation, utility trenches and other below-slab excavations should be backfilled with structural compacted fill in accordance with the project specifications.

Design (Conventional Slab-on-Grade)

The floor of the proposed building is recommended to be designed as a conventional slab-on-grade where the floor slab independent of the foundations. The at-grade floor slab may be designed based on a maximum modulus of subgrade reaction (ks) of 175 pounds per cubic inch (pci), supported on a properly prepared subgrade consisting of a minimum 2-foot structural fill layer. If desired, the floor slab may be poured monolithically with foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. 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.

The floor slab is recommended to be underlain by a 4-inch-thick layer of granular base, which is not included as a portion of the recommended structural fill layer recommended in Section 7.2 Building Area. A 15-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.) and control moisture through the floor slab. It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture. The vapor retarder is recommended to be in accordance with ASTM E 1745, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch-thick layer of sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Settlement

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

6.6 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 is expected to consist of existing on-site soil that exhibit a very low expansion potential. An R-value of 30 has been assumed in the preparation of the pavement design. It should, however, be recognized that the City of La Quinta 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	4	6	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 or less to control shrinkage cracking. Load transfer reinforcing is recommended at construction joints perpendicular to traffic flow if construction joints are not properly keyed. In this event, ¾-inch diameter smooth dowel bars, 18 inches in length placed at 12 inches on-center are recommended where joints are perpendicular to the anticipated traffic flow. Expansion joints are recommended only where the pavement abuts fixed objects such as light standard foundations. Tie bars are recommended at the first joint within the perimeter of the concrete pavement area. Tie bars are recommended to be No. 4 bars at 42-inch on-center spacings and at least 48 inches in length.

General Considerations

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

6.7 Recommended Construction Materials Testing Services

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

6.8 Basis of Report

This report is based on Giles' proposal, which is dated October 26 2022 and is referenced by Giles' proposal number 2GEP-2210025. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

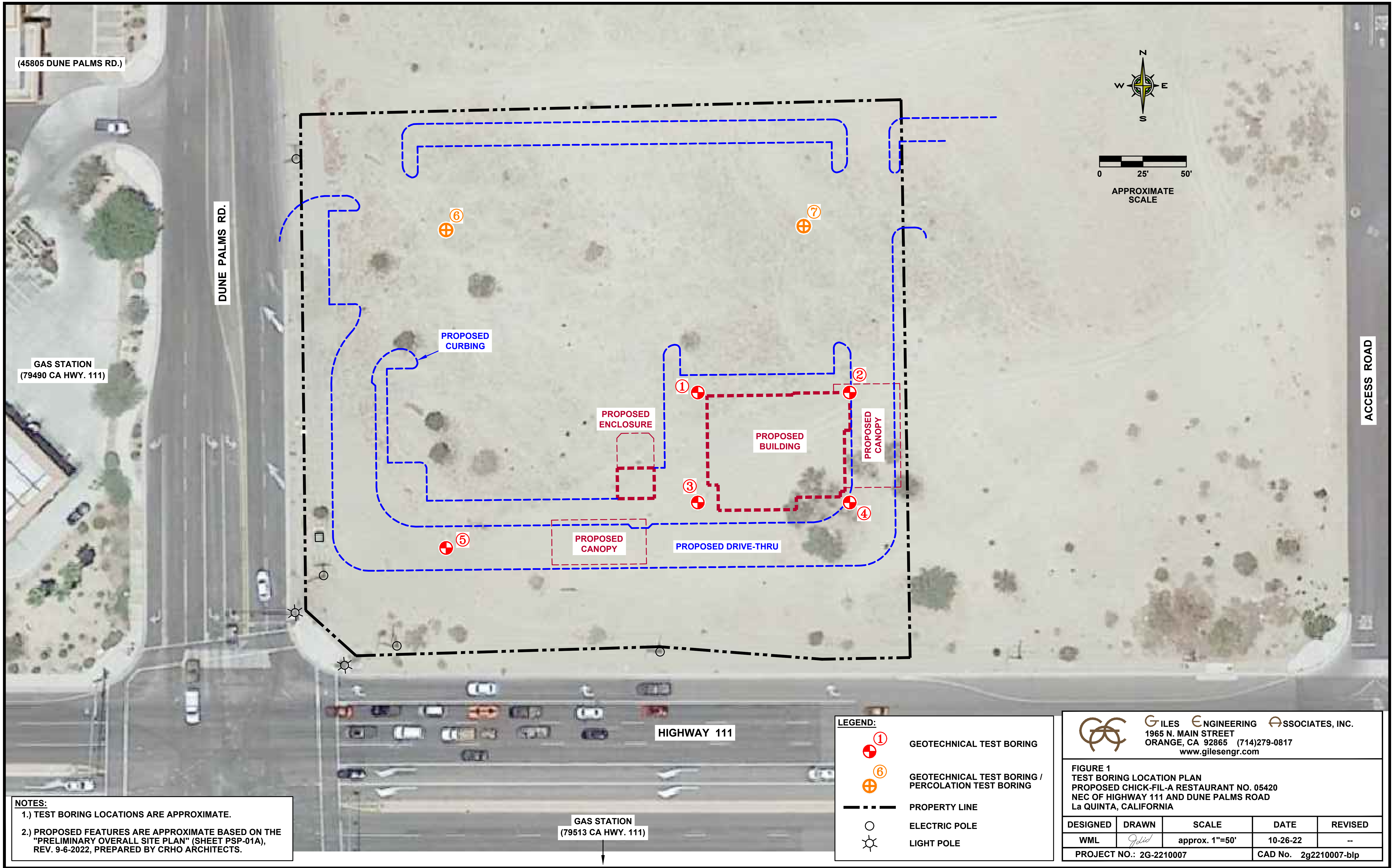
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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



NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "PRELIMINARY OVERALL SITE PLAN" (SHEET PSP-01A), REV. 9-6-2022, PREPARED BY CRHO ARCHITECTS.

LEGEND:

① ◕	GEOTECHNICAL TEST BORING
⑥ ⊕	GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING
▬ ▪ ▪ ▬	PROPERTY LINE
○	ELECTRIC POLE
☼	LIGHT POLE

GILES ENGINEERING ASSOCIATES, INC.
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FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED CHICK-FIL-A RESTAURANT NO. 05420
NEC OF HIGHWAY 111 AND DUNE PALMS ROAD
La QUINTA, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
WML	Jdid	approx. 1"=50'	10-26-22	--

PROJECT NO.: 2G-2210007
CAD No. 2g2210007-blp

BORING NO. & LOCATION:
B-1

SURFACE ELEVATION:
56 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brownish Gray, Silty fine Sand, some mica - Damp (Possible Fill)		55	1-SS	4				2	BDL	P200 = 21%
Brownish Gray, Silty Sand, fine grained - Damp (Native)	5		2-SS	5				1	BDL	P200 = 14%
Brownish Gray to Dark Gray		50	3-SS	4				2	BDL	
	10	45	4-SS	10				2	BDL	
Brownish Gray, fine Sandy Silt, some mica - Moist	15	40	5-SS	10				3	BDL	
	20	35	6-SS	15				2	BDL	

Boring Terminated at about 21.5 feet (EL. 34.5')

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

Remarks:

SS = Standard Penetration Test
Drilling Equipment Used: Hollow-Stem Auger; 8-inch Diameter
Elevations based on Google Earth
PID = Photoionization Detector in ppm
BDL = Below Detectable Limits

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

BORING NO. & LOCATION: B-2	
SURFACE ELEVATION: 53 feet	
COMPLETION DATE: 11/21/22	
FIELD REP: M. KORDAVI	

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brownish Gray, poorly graded Sand, fine grained, some Silt - Dry (Possible Fill)			1-SS	3				1	BDL	
		50								
Brownish Gray, fine Sandy Silt - Damp (Native)			2-SS	4				3	BDL	P200 = 81%
	5									
			3-SS	4				5	BDL	P200 = 78%
		45								
Some Clay, trace fine Gravel	10		4-SS	6				9	BDL	
		40								
Less Clay content	15		5-SS	9				4	BDL	
		35								
Brownish Gray, poorly graded Sand with Silt, fine grained, some mica - Moist	20		6-SS	11				3	BDL	

Boring Terminated at about 21.5 feet (EL. 31.5')

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

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

BORING NO. & LOCATION:
B-3

SURFACE ELEVATION:
54 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brownish Gray, Silty Sand, some mica, fine grained - Damp (Possible Fill)										
Brownish Gray, Silty Sand, fine grained - Damp (Native)			1-SS	3				2	BDL	P200 = 12%
		50	2-SS	5				1	BDL	P200 = 20%
	5									
			3-SS	5				2	BDL	P200 = 21%
		45								
	10		4-SS	8				2	BDL	
		40								
Gray, poorly grained Sand with Silt, fine grained - Damp	15		5-SS	14				2	BDL	
		35								
Gray, poorly graded Sand, fine grained - Damp	20		6-SS	12				2	BDL	

Boring Terminated at about 21.5 feet (EL. 32.5')

Water Observation Data

▽ Water Encountered During Drilling: None
▼ Water Level At End of Drilling:
▨ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
■ Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

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

BORING NO. & LOCATION:
B-4

SURFACE ELEVATION:
56 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brownish Gray, Silty Sand, fine grained, some mica - Damp (Possible Fill)		55	1-SS	5				1	BDL	P200 = 20%
Brownish Gray, Silty Sand, fine grained, some mica - Damp (Native)	5	50	2-SS	5				3	BDL	P200 = 31%
			3-SS	5				3	BDL	
Brownish Gray, fine Sandy Silt, some mica - Moist	10	45	4-SS	5				4	BDL	
	15	40	5-SS	10				3	BDL	
Gray, poorly graded Sand, fine grained - Damp	20	35	6-SS	13				2	BDL	

Boring Terminated at about 21.5 feet (EL. 34.5')

Water Observation Data

▽ Water Encountered During Drilling: None
▼ Water Level At End of Drilling:
▨ Cave Depth At End of Drilling:
▼ Water Level After Drilling:
■ Cave Depth After Drilling:

Remarks:

SS = Standard Penetration Test

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

BORING NO. & LOCATION:
B-5

SURFACE ELEVATION:
55 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Brownish Gray, Silty Sand, fine grained - Damp (Possible Fill)			1-SS	7				1	BDL	
Brownish Gray, Silty Sand, fine grained - Damp (Native)			2-SS	7				2	BDL	
	5	50								
Brownish Gray, fine Sandy Silt, some mica - Moist			3-SS	5				4	BDL	
	10	45								
			4-SS	8				5	BDL	

Boring Terminated at about 11.5 feet (EL. 43.5')

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

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

BORING NO. & LOCATION:
B-6

SURFACE ELEVATION:
58 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Light Gray, Silty Sand, fine grained - Damp (Possible Fill)		57.5								
			1-SS	3				1	BDL	
	2.5	55.0								
Light Brown, Silty Sand, fine grained - Damp (Native)										P200 = 21%
			2-SS	4				2	BDL	
	5.0									

Boring Terminated at about 5 feet (EL. 53')

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

Remarks:

SS = Standard Penetration Test

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

BORING NO. & LOCATION:
B-7

SURFACE ELEVATION:
53 feet

COMPLETION DATE:
11/21/22

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #05420

NEC OF HWY 111 AND DUNE PALMS ROAD
LA QUINTA, CA

PROJECT NO: 2G-2210007

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Gray, Silty Sand, fine grained, some mica - Damp (Possible Fill)			1-SS	3				1	BDL	
		50								
Brownish Gray, fine Sandy Silt, some mica - Moist (Native)			2-SS	5				11	BDL	
	5									
			3-SS	3				6	BDL	
		45								
Some Clay			4-SS	5				29	BDL	P200 = 94%
	10									

Boring Terminated at about 10 feet (EL. 43')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	SS = Standard Penetration Test
▼	Water Level At End of Drilling:	
▨	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
■	Cave Depth After Drilling:	

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

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.

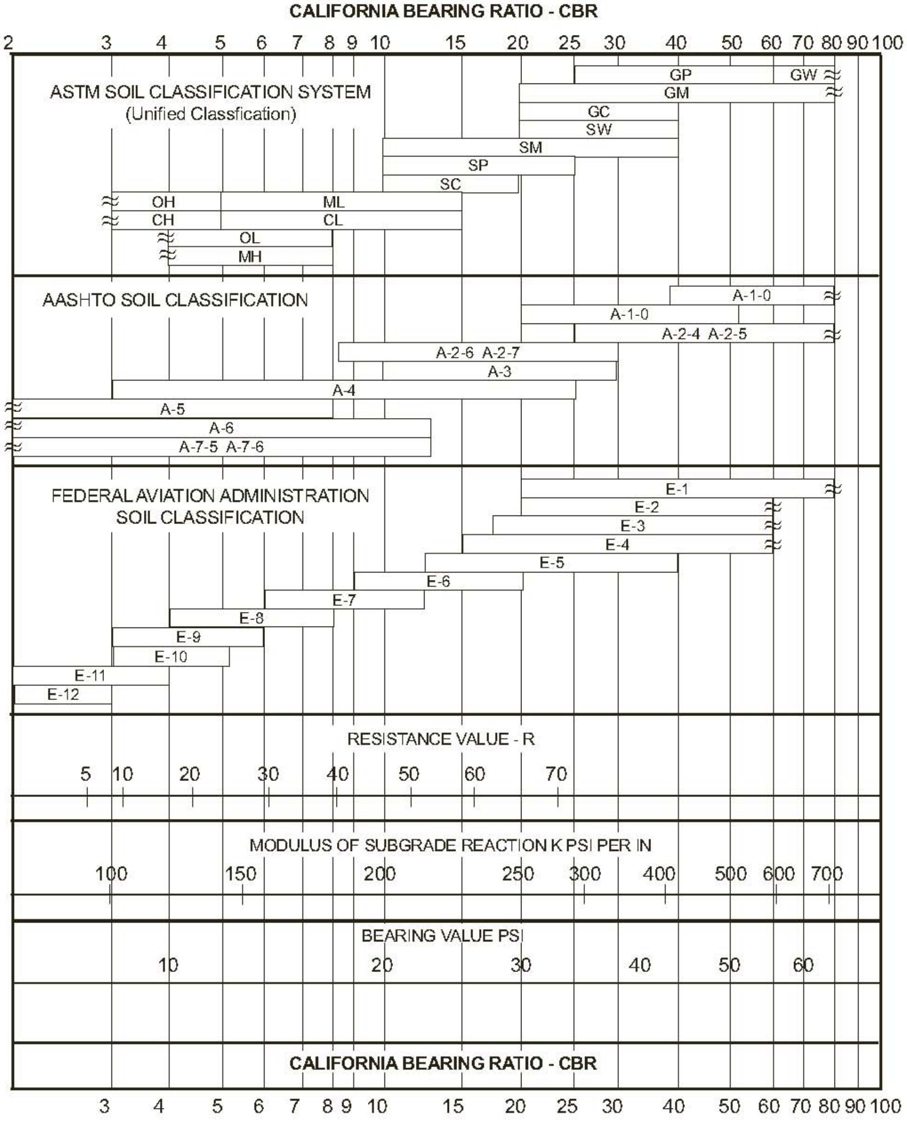
Laboratory Testing

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.

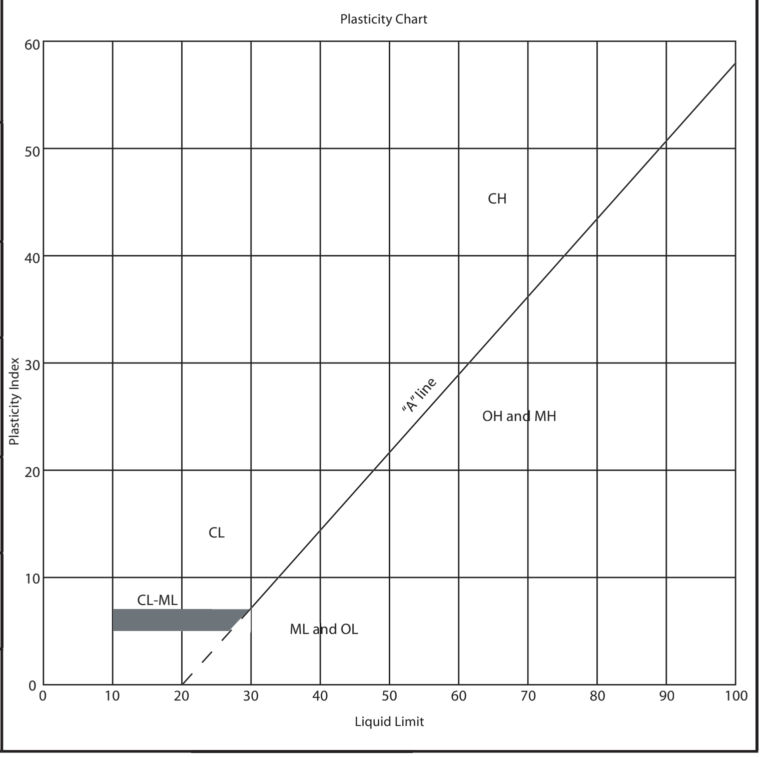
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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	 Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

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

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

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