APPENDIX C
Geotechnical Analysis

● **RED**

Geotechnical Engineering
Exploration and Analysis

Proposed Chick-fil-A Restaurant #05524
West 13th Street & Centre City FSU SWC
W. 13th Avenue & S. Pine Street
Escondido, California

Prepared for:

Chick-fil-A, Inc.
Irvine, California

Prepared by:

Giles Engineering Associates, Inc.

February 12, 2024
Project No. 2G-2303005

GILES
ENGINEERING ASSOCIATES, INC.

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

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

February 12, 2024

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

Attention: Ms. Jessika Guerrero
Restaurant Development Services

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Chick-fil-A Restaurant #05524
West 13th Street & Centre City FSU
SWC W. 13th Avenue & S. Pine Street
Escondido, California
Project No. 2G-2303005

Dear Ms. Guerrero:

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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TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS
PROPOSED CHICK-FIL-A RESTAURANT #05524
WEST 13TH STREET & CENTRE CITY FSU
SWC W. 13TH AVENUE & S. PINE STREET
ESCONDIDO, CALIFORNIA
PROJECT NO. 2G-2303005

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED CHICK-FIL-A RESTAURANT #05524
WEST 13TH STREET & CENTRE CITY FSU
SWC W. 13th AVENUE & S. PINE STREET
ESCONDIDO, CALIFORNIA
PROJECT NO. 2G-2303005

EXECUTIVE SUMMARY OUTLINE

The executive summary is provided solely for the purpose 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 C is recommended for seismic design considerations.
- Our review of the Geology of the Geologic Map of the Oceanside 30' x 60' Quadrangle, (Kennedy & Tan, 1977), indicates the site is underlain by Old Alluvial flood-plain deposits, undivided, late to middle Pleistocene, (Qoa). Additionally, the subject site is located near the Granodiorite of Woodson Mountain (Kwm) formation.
- Fill and possible fill soils were encountered up to 3 feet in thickness. Fill material consists of moist to very moist, loose to medium dense in relative density clayey sand, fine to medium grained, some coarse, and medium stiff to stiff sandy silt and clay.
- Native soils consist of moist, very stiff sandy clay, fine to medium sand. Below the fill and native soils, Granite material was encountered in the borings consisting fine to medium grained, some coarse, weakly weathered and cemented, some mica, with various amount of fine gravel and some silt. Sandy Siltstone layer was encountered in Test Boring 3 above the Granite material.
- Groundwater was not encountered during our field exploration in the test borings, except for Test Boring 3, which was measured at approximately 13.9 feet below ground. However, it is our opinion that the groundwater may be perched groundwater encountered within a more granular layer in between the Sandy Siltstone and Granite bedrock.
- Tested onsite soils generally possess a **low expansion potential**.
- Tested on-site soils have **moderately 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 include new concrete walkways, parking stalls, driveways, drive thru lane, canopies, and trash enclosure.
- Due to the variable strength characteristics of the near surface onsite soils 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 1 foot structural fill layer below the foundations and floor slab with the existing soils proofrolled for fill areas, or proofrolled following the required cut, with removal of any unstable materials. The existing soils are considered suitable for foundation and floor support with the recommended 1-foot structural fill layer and for pavement support with recommended proofroll and geotechnical inspection/testing. The existing soils exposed for fill placement or soils exposed after the required excavation to obtain subgrade elevation should be, in addition to the required proofrolling activities, examined by the geotechnical engineer to document that the soils are suitable for building support. Deeper fills may be discovered during construction.

Building Foundation

- Shallow spread footing foundation systems or turned-down slabs may be designed for a maximum, net allowable soil pressure of 3,500 psf soil bearing pressure supported on newly placed structural compacted fill.
- The proposed canopies may be supported by into compacted fill (footings), or native material (drilled pier footing) encountered within our borings verified in the field by the Geotechnical Engineer of Record during excavation, the axial (downward) skin friction (side resistance) resistance was determined to be 150 psf from our field data obtained during our field investigations at the site. This capacity is in addition to the allowable soil bearing pressure of 3,500 psf for a shallow footing foundation, or 3,500 psf for drilled pier footing. 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.

Building Floor Slab

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

New Pavement

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

RED - This site has been given a RED designation due to shallow bedrock encountered, and difficulty of excavation potential, especially for deeper utility trenches.

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.

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

Giles conducted a *Phase 1 Environmental Site Assessment* for the subject site. The results of that assessment were provided under separate cover (2E-2303008). Additionally, an Asbestos Identification Survey was also conducted and presented under Report No. 2E-2303008, dated April 26, 2023.

2.0 SITES AND PROJECT DESCRIPTION

2.1 Site Description

The proposed Chick-fil-A development is located at 515 West 13th Avenue, SWC W. 13th Avenue and S. Pine Street, in the City of Escondido, San Diego County, California. The site is currently occupied by a vacant restaurant building with parking areas. The site is also developed with landscape with grass, trees, and shrubs. The site is bounded on the northwest by W. 13th Avenue, on the northeast by S. Pine Street, on the south by a multiple mobil homes park, and on the west by more parking lot and few comercial structures.

Site topography is fairly level, with Elevations range between 657 and 658 across the site. The subject property is situated at approximately latitude of 33.1075° North and longitude of -117.0793° West.

The site parking lot is currently covered with asphalt pavement along with curbs and few landscape planters that contain shrubs and trees. Other existing site improvements include lighting poles and underground utilities.

2.2 Proposed Project Description

The proposed development includes the construction of a new, single-story Chick-fil-A restaurant building with drive thru lanes to be located along the northern area of the proposed building (Figure 1). The existing building is planned to be demolished to construct the new building. Although detailed building plans are not yet ready for our review, the new building will be a single-story wood-frame structure, with no basement or underground levels, to be located within the northeastern area of the property. We were not provided with specific loading information for this project at the time of this report; however, based on previous Chick-fil-A projects, we expect maximum combined dead and live loads supported by the bearing walls and columns of 2 to 3 kips per lineal foot (klf) and 40 to 50 kips, respectively. The live load supported by the floor slab is expected to be a maximum of 100 pounds per square foot (psf).

Other planned improvements include new parking lot, menu board signs, outdoor dining area, concrete walkways and planter areas, new canopies, and a trash enclosure. The parking lot improvement within the property will include curbs and gutters, and underground utilities.

Giles has been provided with conceptual civil plans, and in particular the Conceptual Grading Plan, dated November 16, 2023. The planned finished floor elevation for the proposed building is El. 660.10 ft. Therefore, it is anticipated grading for this development will include only minor cut and fill 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 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 occupied as a vacant building used as a former restaurant with parking lot. Based on historical images, the existing structure was first observed in 1980. A previous small commercial structure was present in 1939.

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration consisted of the drilling of eight (8) exploratory test borings to depths of about 5 to 21.5 feet below existing ground surface. The borings were drilled with a 6-inch diameter, hollow-stem auger rig. The approximate test boring locations are shown in the Test Boring Location Plan (Figure 1). The Test Boring Location Plan and Test Boring Logs (Records of Subsurface Exploration) are enclosed in Appendix A. Field and laboratory test procedures and results are enclosed in Appendix B and C, respectively. The terms and symbols used on the Test Boring Logs are defined on the General Notes in Appendix D.

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 containers and transported to our laboratory for testing.

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

At the conclusion of drilling activities, each borehole was backfilled. Even with this service, however, it is important to note that some boreholes backfill settlement or expansion can and will occur over time. This settlement/expansion can create a hazard and should be carefully monitored by the client and/or property owner. The settlement/expansion can lead to the formation of a “trip joint” representing a threat of injury to persons or animals utilizing or accessing the subject property. Giles has not included a cost for monitoring borehole settlement/expansion after the initial drilling activities and will not be performing this service.

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 A of this report.

Pavement

Existing pavement encountered within our test borings consisted of approximately 3 to 4-inch-thick asphalt concrete without any aggregate base. Based on our visual observation, the existing asphalt pavement is in poor to fair condition.

Geology

Our review of the Geology of the Geologic Map of the Oceanside 30’ x 60’ Quadrangle, (Kennedy & Tan, 1977), indicates the site is underlain by Old Alluvial flood-plain deposits, undivided, late to middle Pleistocene, (Qoa). Additionally, the subject site is located near the Granodiorite of Woodson Mountain (Kwm) formation.

Soil

On-site soils encountered during our subsurface exploration consisted generally of fill and possible fill materials and native soils. Fill and possible fill soils were encountered up to 3 feet in thickness. Fill material consists of moist to very moist, loose to medium dense in relative density clayey sand, fine to medium grained, some coarse, and medium stiff to stiff sandy silt and clay.

Native soils consist of moist, very stiff sandy clay, fine to medium sand.

Bedrock

Below the fill and native soils, Granite material was encountered in the borings consisting fine to medium grained, some coarse, weakly weathered and cemented, some mica, with various amount of fine gravel and some silt. Sandy Siltstone layer was encountered in Test Boring 3 above the Granite material.

Groundwater

Groundwater was not encountered during our field exploration, except for Test Boring 3, which was measured at approximately 13.9 feet below ground. However, it is our opinion that the groundwater may be perched groundwater encountered within a more granular layer in between the Sandy Siltstone and Granite bedrock. Fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site can also cause fluctuations of local or shallow perched groundwater levels.

3.3 Percolation Testing

It is our understanding that a below grade storm water infiltration system is being considered for the site. Although specific details were not available, we chose a spot within the proposed parking lot to screen the site for stormwater infiltration purposes.

One percolation test was conducted in Test Borings B-4. The percolation test was performed in accordance with United States Bureau of Reclamation USBR 7300-89 guidelines. The in-situ percolation tests involved the drilling of a test boring by utilizing a hollow-stem auger rig with an outside diameter of approximately 6 inches to a depth of about 10 feet.

The approximate percolation test boring location is shown on the Test Boring Location Plan (Figure 1). A slotted 2-inch diameter pvc pipe was installed inside the test boring with gravel placed below and on the sides of the perforated pipe. The percolation test involved presoaking the boring, filling the test hole with water, recording the drop in water surface with time, and refilling the hole with water following each reading. Readings of the change in height of water were measured and recorded at intervals of about 10 to 30 minutes. A series of readings were taking by using the falling head method until water reached steady conditions. Porchet formula was used to estimate the infiltration rate as follows:

Design Infiltration Rate = $\Delta H\ (60*r)/\ \Delta t*(r + 2*H_{AVE})$
Where:

$\Delta H = H_o - H_f$ (inches)
H_o = Initial water level; H_f = Final water level
r = radius of hole (inch)
Δt = time interval (mins)
$H_{AVE} = (H_o + H_f)/\ 2$

The results of the percolation test are presented on the following Table 1.

Table 1 – Percolation Test Results

Percolation Test Results			
Test No.	**Test Depth (Feet) below Existing Surface Grade**	**Infiltration Rate (In/hr)**	**Soil Type**
B-4	10	0.01	Granite

In/hr = inch per hour

It should be noted that the infiltration rate of the on-site soil represents a specific area and depth tested and may fluctuate throughout other areas of the site. The infiltration rate noted above has not been reduced to account for a factor of safety.

Please note that based on our percolation test results and drilling exploration, it is Giles' opinion that an on-site stormwater infiltration system will not be suitable for this site due to the nature of the impermeable on-site soils below and shallow bedrock. If an infiltration system is still required for this development, artificial systems may need to be designed by a civil engineer and to determine the type of system to use.

4.0 LABORATORY TESTING

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

In Situ Moisture and Density

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

Sieve Analysis

Representative samples were used to perform a No. 200 Sieve analysis to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 1140. The result of the Passing No. 200 Sieve is presented in the Test Boring Logs enclosed in Appendix A.

Expansion Index

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite bulk sample collected within the upper soils from all the Test Borings was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829. The result of our expansion index (EI) test indicates that the near surface sample has a low expansion potential (EI= 38).

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 collected during our field exploration and used to determine the corrosion potential for buried ferrous metal conduits and the concentrations present of water-soluble sulfate which could result in chemical attack of cement. The following table presents the results of our laboratory testing.

Parameter	**Bulk Sample 1 to 5 feet**
pH	7.3
Chloride	258 ppm
Sulfate	0.024%
Resistivity	2,200 ohm-cm

The chloride content of the near-surface soils was determined for a selected sample in accordance with California Test Method No. 422. The results of this test indicated that tested on-site soil has a **moderate** exposure to chloride. The results of limited in-house testing of soil pH and resistivity were determined in accordance with California Test Method No. 643 and indicated that on-site soil is **neutral** with respect to pH and soil resistivity was found to possess a **moderately degree of corrosivity**.

These test results have been evaluated in accordance with criteria established by the Cast Iron Pipe Research Association, Ductile Iron Pipe Research Association, the American Concrete Institute and the National Association of Corrosion Engineers. The test results on a near surface sample from the site generally indicate that tested on-site soils have **moderately corrosive potential** when in contact with ferrous materials. We recommend that a corrosion engineer review these results in order to provide specific recommendations for corrosion protection as well as appropriate recommendations for other types of buried metal structures.

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample in accordance with California Test Method No. 417. Our laboratory test data indicated that near surface soils contain approximately 0.024 percent of water-soluble sulfates. Based on the 2022 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318. Therefore, according to Table 4.3.1 of the ACI 318, a **low exposure to sulfate corrosivity** can be expected for concrete placed in contact with the tested on-site soils. No special sulfate resistant cement is considered necessary for concrete which will be in contact with the tested on-site soils.

5.0 GEOLOGIC AND SEISMIC HAZARDS

5.1 Active Fault Zones

Research of available maps published by the Escondido General Plan (2011), Figure 4.6-1, Regional Faults, the subject site is not located within any active fault zone. The site may however be subject to strong ground shaking during seismic activity.

5.2 Seismic Hazard Zones

Based on the Escondido General Plan, Figures 4.6-3 and 4.6-4, the site is located outside the potential liquefaction and landslide areas. Furthermore, due to our recent field exploration, very dense granitic bedrock and lack of groundwater, except for a perched water condition in Test Boring B-3, it is our opinion the liquefaction potential for the site is considered to be low.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

According to the maps of known active fault near-source zones to be used with the CBC, the Newport Inglewood Connected alt 2 and Rose Canyon Faults are the closest known active faults and are both located about 15.6 miles to the site. The Newport-Inglewood Fault would probably generate the most severe site ground motions at the site with an anticipated maximum moment magnitude (Mw) of 7.5.

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 2022 *CBC* and *ASCE 7-16*.

Table 2 – Recommended Seismic Design Parameters

CBC 2022, Earthquake Loads	
Site Class Definition (Table 20.3-1 from ASCE 7-16)	C
Mapped Spectral Response Acceleration Parameter, S_s (for 0.2 second)	0.886
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second)	0.324
Site Coefficient, F_a short period	1.2
Site Coefficient, F_v 1-second period	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS}	1.063
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1}	0.487
Design Spectral Response Acceleration Parameter, S_{DS}	0.709
Design Spectral Response Acceleration Parameter, S_{D1}	0.324
MCEG Peak Ground Acceleration adjusted for site class effects, PGA_M	0.458

6.2 Site Development and Construction Recommendations

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

Site Clearing

Clearing and demolition operations should include the removal of all landscape vegetation and existing structural features such as existing footings and slabs, sidewalks, asphaltic concrete pavement, concrete curb and gutters within the area of the proposed new building and site improvements. Existing pavement within areas of proposed development should be removed or processed to a maximum 3-inch size and stockpiled for use as compacted fill or stabilizing material for the new development. Processed asphalt may be used as fill, sub-base course material, or subgrade stabilization material beyond the building perimeter. Processed concrete or existing base may be used as fill, sub-base course material, or subgrade stabilization material both within and outside of the building perimeter.

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

Existing Utilities

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

Building Area

Due to the variable strength characteristics of the near surface onsite soils 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 1 foot structural fill layer below the foundations and floor slab with the existing soils proofrolled for fill areas, or proofrolled following the required cut, with removal of any unstable materials. The existing soils are considered suitable for foundation and floor support with the recommended 1-foot structural fill layer and for pavement support with recommended proofroll and geotechnical inspection/testing. The existing soils exposed for fill placement or soils exposed after the required excavation to obtain subgrade elevation should be, in addition to the required proofrolling activities, examined by the geotechnical engineer to document that the soils are suitable for building support. Deeper fills may be discovered during construction. Depending on examination by the geotechnical engineer, some additional over-excavation may be required. 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 near optimum moisture content and then compacted 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.

Proofroll and Compact Subgrade

Following over-excavation and lowering of site grades, where necessary, the subgrades within the proposed building pad and any new pavement area should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded dump truck to detect very loose/soft yielding soil which should be removed to a stable subgrade. Following proofrolling and completion of any necessary overexcavation, the subgrades should be scarified to a depth of at least 8 to 12 inches, moisture conditioned or air dried as recommended, and recompacted to at least 90 percent of the Modified Proctor maximum dry density. In accordance with the enclosed Guide Specifications and in the event that new pavement is constructed within the site, the top 12 inches of the pavement subgrade soils should be compacted to at least 95 percent of the Modified Proctor maximum density, with the underlying fill materials compacted to 90 percent of the Modified Proctor maximum density. Low areas and excavations may then be backfilled in lifts with suitable low expansive (EI<50) structural compacted fill.

The selection, placement and compaction of structural fill should be performed in accordance with the project specifications. The Guide Specifications included in Appendix D (Modified Proctor) of this report should be used as a minimum in developing the project specifications. The need may arise to recompact the floor slab and pavement subgrades immediately prior to construction due to the effects of weather and construction traffic on a previously prepared subgrade.

Reuse of On-site Soil

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

Import Structural Fill

Any soil imported to the site (if required) for use as structural fill should consist of low expansive soils (EI less than 50). Material designated for import should be submitted to the project geotechnical engineer no less than three working days prior to placement for evaluation.

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

Fill Placement

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

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

The upper 12 inches of subgrade material within the pavement area should be moisture conditioned near the soil’s optimum moisture content, and then compacted in place to at least 95 percent of the Modified Proctor maximum density, and in accordance with the enclosed “Guide Structural Fill Specifications”. A representative of the project geotechnical engineer should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Soil Excavation

All excavations must be performed in accordance with CAL-OSHA requirements, which is the responsibility of the contractor. Shallow excavations may be adequately sloped back at a uniform 1:1 (h:v) slope gradient in its entirety to a maximum height of 5 feet. All other excavations should be shored or braced. Water should not be allowed to pond on top of the excavation nor to flow towards it. The soils exposed in the cut slopes should be inspected during excavation by a representative of this firm, so that modifications of the slopes can be made if any variations in the soil conditions occur.

6.3 Foundation Recommendations

Vertical Load Capacity – Shallow Foundation

Upon completion of the building pad preparation, the proposed structure may be supported by a shallow foundation system. Foundations may be designed for a maximum, net, allowable soil-bearing pressure of 3,500 pounds per square foot (psf). Minimum foundation widths for walls and columns should be 16 and 24 inches, respectively, regardless of the calculated soil bearing pressure. The maximum bearing value applies to combined dead and live loads. The recommended allowable soil bearing pressure may be increased by one-third for short term wind and/or seismic loads.

Estimated Foundation Settlement

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

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 competent native soil and/or newly placed compacted fill soil. An allowable passive earth pressure of 375 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 consideration. Interior footings may be supported at nominal depth below the floor. All footings must be protected against weather and water damage during and after construction and must be supported with suitable bearing materials.

Canopy Foundation

For this foundation system embedded into compacted fill (footings), or native material (drilled pier footing) encountered within our borings verified in the field by the Geotechnical Engineer of Record during excavation, the axial (downward) skin friction (side resistance) resistance was determined to be 150 psf from our field data obtained during our field investigations at the site. This capacity is in addition to the allowable soil bearing pressure of 3,500 psf for a shallow footing foundation, or 3,500 psf for drilled pier footing. 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 150 psf. Capacities for other pile types, dimensions, and lengths can be provided upon request.

For uplift resistance, an average allowable side resistance of 75 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.

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. An uncased pier excavation should not be approached, as it could rapidly cave. Concrete is recommended to be placed in accordance with "state-of-the-practice" procedures under engineering-controlled conditions as noted below. Drilled pier construction should be done in accordance with local codes, and other pertinent requirements.

Pier 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 35 pcf, 55 pcf, and 375 pcf may be used for soil parameters, respectively. Reduction factors may be needed for group action for lateral capacities, dependent on the configuration of pier groups and the direction of applied lateral loads. The maximum recommended allowable passive pressure is 2,500 psf.

Reinforcing

The design of the foundation systems as well as determination of the actual quantity of steel reinforcing and the footing dimensions should be performed by the project structural engineer.

Bearing Material Criteria

Soil suitable to serve as the foundation structural fill subgrade should exhibit at least a loose relative density, based on the 3,500 psf allowable soil bearing pressure, should possess at least a loose relative density (average N value of at least 10) for non-cohesive soils, and an unconfined compressive strength of 1.75 tsf for cohesive soils. For design and construction estimating purposes, suitable bearing soils are expected to be encountered at nominal foundation depths following the recommended site preparation activities. However, field testing by the Geotechnical Engineer within the foundation bearing soils is recommended to document that the structural fill subgrade soils and therefore the foundation supporting soils possess the minimum strength parameters noted above. Testing may consist of Dynamic Cone Penetration tests (per ASTM Special Publication 399), nuclear gauge, sand cone tests, or other tests as deemed suitable by the geotechnical engineer. 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.

6.4 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 of 1 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

The floor of the proposed building may be designed and constructed as a conventional slab-on-grade supported on a properly prepared subgrade. If desired, the floor slab may be poured monolithically with perimeter foundations where the foundations consist of thickened sections thereby using a turned-down slab construction technique. The minimum slab reinforcing for geotechnical considerations is recommended to consist of No. 3 rebar at 18 inches on center, each way. Based on the recommended reinforcing and the assumed live loading, the slab is recommended to be a minimum of 4 inches in thickness. A qualified structural engineer should perform the actual design of the slab to ensure proper thickness and reinforcing. If desired, a Subgrade Modulus of 175 pci may be used for floor slab design.

The floor slab is recommended to be underlain by a 4-inch-thick layer of granular material. A minimum 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.). It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture on future flooring materials. The vapor retarder is recommended to be in accordance with ASTM E 1745, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. The sheets of the vapor retarder material should be evaluated for holes and/or punctures prior to placement and the edges overlapped and taped. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of coarse sand (Sand Equivalent>20) approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch-thick layer of coarse sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Movements

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

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

General Considerations

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

6.6 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.

6.7 Basis of Report

This report is based on Giles' proposal, which is dated March 21, 2023 and is referenced by Giles' proposal number 2GEP-2303010. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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

GILES ENGINEERING ASSOCIATES, INC.
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FIGURE 1
TEST BORING LOCATION PLAN
CHICK-FIL-A RESTAURANT NO. 05524
SWC OF W. 13th AVENUE AND S. PINE STREET
ESCONDIDO, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
WML		approx. 1"=40'	06-23-23	–

PROJECT NO.: 2G-2303005
CAD No. 2g2303005-blp

LEGEND:

①	GEOTECHNICAL TEST BORING
④	GEOTECHNICAL TEST BORING / PERCOLATION TEST BORING
— · · —	PROPERTY LINE
○	ELECTRIC POLE
☼	LIGHT POLE
☒	CATCH BASIN
▽	FIRE HYDRANT

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "PRELIMINARY SITE PLAN" (SHEET PSP-07), REV 2-28-2023, PREPARED BY CRHO ARCHITECTS.

3.) PROPERTY LINES ARE APPROXIMATE BASE ON A SAN DIEGO COUNTY GIS AERIAL.

4.) EXISTING BUILDING IS APPROXIMATE BASED ON THE "SITE PLAN" (SHEET P01), DATED 2-8-2021, PROVIDED BY THE CLIENT.

BORING NO. & LOCATION:
B-1

SURFACE ELEVATION:
657 feet

COMPLETION DATE:
04/11/23

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #5524

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

PROJECT NO: 2G-2303005

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of Asphaltic Concrete, no Aggregate Base										
Orange brown, Clayey Sand, fine to medium grained, some coarse - Moist (Possible Fill)		655	1-SS	29				7	BDL	
Black and orange brown mottled, Granite, weathered, fine to coarse grained, weakly cemented - Moist (Native)			2-SS	50/6"				7	BDL	
	5									
Trace fine Gravel, some Silt.			3-SS	50/5"				10	BDL	
		650								
	10		4-SS	50/5"				7	BDL	

Boring Terminated at about 11.5 feet (EL. 645.5')

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

Remarks:

SS = Standard Penetration Test
Drilling Equipment: Hollow-Stem Auger; 8-inch diameter
BDL = Below Detection Limit
Elevations based on Google Earth (approximate)

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

BORING NO. & LOCATION: B-2	# TEST BORING LOG	
SURFACE ELEVATION: 658 feet	PROPOSED CHICK-FIL-A RESTAURANT #5524	**GILES ENGINEERING ASSOCIATES, INC.**
COMPLETION DATE: 04/11/23	WEST 13TH AND CENTRE CITY FSU SWC W. 13TH AVENUE AND S. PINE STREET ESCONDIDO, CA	
FIELD REP: M. KORDAVI	PROJECT NO: 2G-2303005	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base										
Brown, fine Sandy Silt, trace Clay, some mica - Very Moist (Possible Fill)			1-SS	5				16	BDL	
		655								
Brown, Sandy Clay, fine to medium Sand - Moist (Native)			2-SS	22		>4.5		13	BDL	
Grayish brown, Granite, fine to medium grained, some coarse, some mica, weakly cemented - Moist	5									
			3-SS	50/4"				4	BDL	
		650								
	10		4-SS	50/4"				3	BDL	
		645								
	15		5-SS	50/5"				5	BDL	

Boring Terminated at about 16.5 feet (EL. 641.5')

	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	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 657 feet	PROPOSED CHICK-FIL-A RESTAURANT #5524	
COMPLETION DATE: 04/11/23	WEST 13TH AND CENTRE CITY FSU SWC W. 13TH AVENUE AND S. PINE STREET ESCONDIDO, CA	
FIELD REP: M. KORDAVI	PROJECT NO: 2G-2303005	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base										
Dark brown, fine Sandy Silt, some mica - Moist (Possible fill)		655	1-SS	8				11	BDL	
Grayish brown and white mottled, Granite, weakly weathered, fine to coarse grained- Moist (Native)			2-CS	50/6"				8	BDL	
	5									
Light tan, Sandy Siltstone, some cemented fragments, fine to medium Sand - Moist			3-SS	50/2"				5	BDL	
		650								
More Cemented	10		4-SS	50/3"				2	BDL	
		645								
	∇									
Grayish brown and white mottled, Granite, weakly weathered, fine to medium grained, some mica - Moist	15		5-SS	50/3"				3	BDL	
		640								
More coarse - Very moist to Wet	20		6-SS	50/6"				10	BDL	
Boring Terminated at about 21.5 feet (EL. 635.5')										

Water Observation Data	Remarks:
∇ Water Encountered During Drilling: 13.9 ft.	SS = Standard Penetration Test CS = California Split Spoon
▼ Water Level At End of Drilling:	
▨ Cave Depth At End of Drilling:	
▼ Water Level After Drilling:	
▬ Cave Depth After Drilling:	

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

BORING NO. & LOCATION:
B-4

SURFACE ELEVATION:
657 feet

COMPLETION DATE:
04/11/23

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #5524

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

PROJECT NO: 2G-2303005

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base.										
Dark brown, Sandy Clay, fine to medium Sand, some Silt - Very Moist (Possible Fill)		655	1-SS	6				18	BDL	
Orange brown and white mottled, Granite, fine to coarse grained, cemented material - Moist (Native)			2-SS	50				8	BDL	
	5									
Interbedded dark brown and light tan, Clayey Sandstone - Moist			3-CS	50/5"				13	BDL	
		650								
	10		4-SS	50/4"				5	BDL	
Boring Terminated at about 10.5 feet (EL. 646.5')										

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
CS = California Split Spoon

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:
656 feet

COMPLETION DATE:
04/11/23

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #5524

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

PROJECT NO: 2G-2303005

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base										
Grayish brown, Clayey Sand, fine to medium grained, some coarse, some mica - Moist (Possible Fill)		655	1-SS	13				10	BDL	
Grayish Brown, Granite, fine to coarse grained, some mica, weakly cemented - Moist (Native)			2-SS	50/4"				6	BDL	
	5	650	3-SS	50/4"				5	BDL	
	10	645	4-SS	50/4"				4	BDL	
	15		5-SS	50/5"				4	BDL	

Boring Terminated at about 15.5 feet (EL. 640.5')

	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-6	**SURFACE ELEVATION:** 658 feet
COMPLETION DATE: 04/11/23	**FIELD REP:** M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #5524

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

PROJECT NO: 2G-2303005

GILES ENGINEERING
ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base										
		657.5								
Brown, fine Sandy Clay - Very Moist (Possible fill)			1-SS	4		3.5		23	BDL	
	2.5	655.0								
Grayish brown and white mottled, Granite, fine to coarse grained, cemented - Moist (Native)			2-SS	63				6	BDL	
	5.0									
Boring Terminated at about 5 feet (EL. 653')										

	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

GILES LOG REPORT 2G-2303005.GPJ GILES.GDT 7/6/23

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:
657 feet

COMPLETION DATE:
04/11/23

FIELD REP:
M. KORDAVI

TEST BORING LOG

PROPOSED CHICK-FIL-A RESTAURANT #5524

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

PROJECT NO: 2G-2303005

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 4 inches of Asphaltic Concrete, no Aggregate Base										
Orange brown, Clayey Sand, fine to medium grained - Moist (Possible Fill)	2.5	655.0	1-SS	9				15	BDL	
Orange brown and gray mottled, Granite, fine to medium grained, some coarse, some mica, weakly cemented - Moist (Native)	5.0	652.5	2-SS	50/5"				6	BDL	
Boring Terminated at about 5 feet (EL. 652')										

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-8

TEST BORING LOG

SURFACE ELEVATION:
658 feet

PROPOSED CHICK-FIL-A RESTAURANT #5524

COMPLETION DATE:
04/11/23

WEST 13TH AND CENTRE CITY FSU
SWC W. 13TH AVENUE AND S. PINE STREET
ESCONDIDO, CA

FIELD REP:
M. KORDAVI

PROJECT NO: 2G-2303005

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
Approximately 3 inches of Asphaltic Concrete, no Aggregate Base		657.5								
Dark brown, fine Sandy Clay, some Silt - Very Moist (Possible Fill)	2.5	655.0	1-SS	9		>4.5		15	BDL	
Grayish brown and orange mottled, Granite, fine to medium grained, some coarse, weakly cemented - Moist (Native)	5.0		2-SS	50/5"				5	BDL	
Boring Terminated at about 5 feet (EL. 653')										

	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.

APPENDIX B

FIELD PROCEDURES

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

GENERAL FIELD PROCEDURES

Test Boring Elevations

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

Test Boring Locations

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

Water Level Measurement

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

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

Borehole Backfilling Procedures

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

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

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

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

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

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

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

Bulk Sample (BS)

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

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

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

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

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

Sampling and Testing Procedures

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

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

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

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

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

Moisture Content (w) (ASTM D 2216)

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

Unconfined Compressive Strength (qu) (ASTM D 2166)

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

Calibrated Penetrometer Resistance (qp)

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

Vane-Shear Strength (qs)

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

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

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

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

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

Consolidation Test (ASTM D 2435)

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

Classification of Samples

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

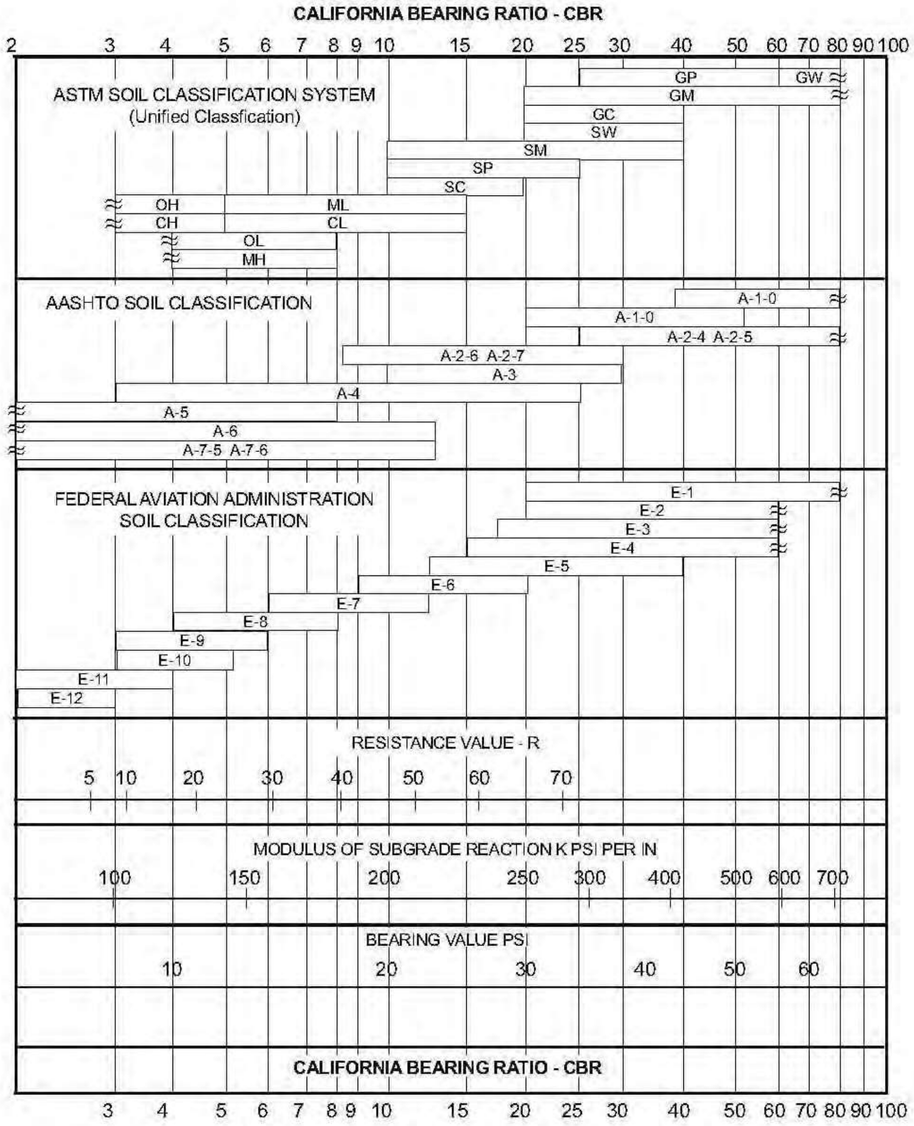
Laboratory Testing

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

California Bearing Ratio (CBR) Test ASTM D-1833

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

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



APPENDIX D

GENERAL INFORMATION

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

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

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

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

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

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

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

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

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

GENERAL COMMENTS

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

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

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

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

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

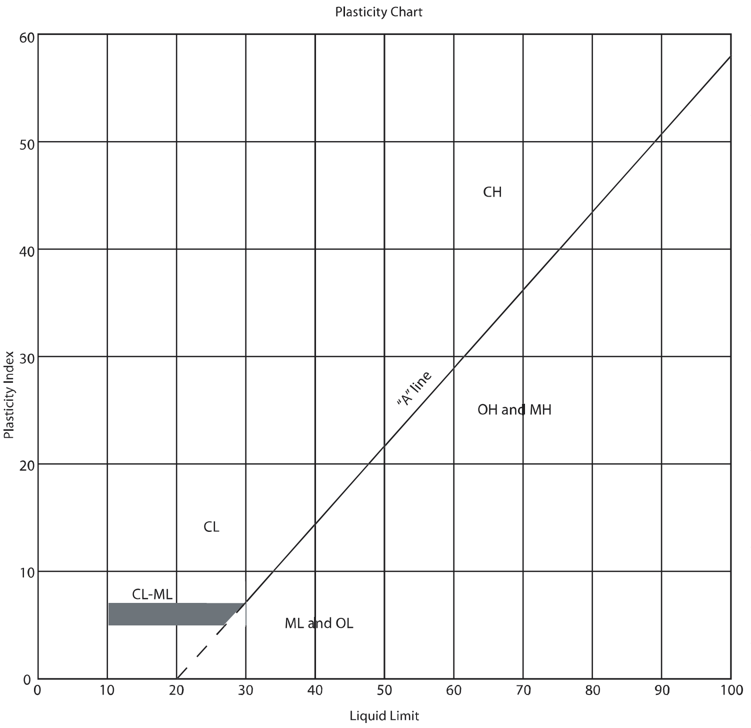
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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

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

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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

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

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are <u>Not</u> Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

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