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GILES ENGINEERING ASSOCIATES, INC.

Preliminary Geotechnical Engineering Exploration and Analysis

Proposed Mixed-Use Development
Lots 3 and 4, Pardee Visitor Center
3510 Valley Centre Drive
San Diego, California

Prepared for:

Carmel Valley Centre Drive, LLC
San Diego, California

April 4, 2014

Project No. 2G-1403001

GILES
ENGINEERING ASSOCIATES, INC.

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

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• Milwaukee, WI
• Orlando, FL

April 4, 2014

Carmel Valley Centre Drive, LLC
7969 Engineer Road, Suite 108
San Diego, California 92111

Attention: Mr. Hunter Oliver
Co-Managing Member

Subject: Preliminary Geotechnical Engineering Exploration and Analysis
Proposed Mixed-Use Development
Lots 3 and 4, Pardee Visitor Center
3510 Valley Centre Drive
San Diego, California
Project No. 2G-1403001

Dear Mr. Oliver:

In accordance with your request and authorization, a *Preliminary Geotechnical Engineering Exploration and Analysis* report has been prepared for the above-referenced project. Conclusions and recommendations developed from the exploration and analysis are discussed in the accompanying report.

We appreciate the opportunity to be of service on this project. If we may be of additional assistance, should geotechnical related problems occur 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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G/Data/Geotechnical/2014/2G-1403001 Mixed-Use Devel, Lots 3&4, Pardee Vistor Ctr, San Diego, CA/Geotechnical Rpt (4-4-14)

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PRELIMINARY GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

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LOTS 3 AND 4, PARDEE VISITOR CENTER
3510 VALLEY CENTRE DRIVE
SAN DIEGO, CALIFORNIA
PROJECT NO. 2G-1403001

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained in this report. The scope of work performed for this report was consistent with the scope of work outlined within Proposal No. 2GP-1403002.

The scope of services authorized for this project included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and a geotechnical engineering analysis to provide criteria for preparing the design of the foundation and floor slab for the proposed development. Geotechnical-related recommendations are also provided for the proposed parking development. Site preparation recommendations are also given; however, those recommendations are only preliminary since the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include the weather before and during construction, the water table at the time of construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development.

2.0 SITES AND PROJECT DESCRIPTION

2.1 Site Description

The subject site (Lots 3 & 4, APN 307-240-03 & 307-240-04), which is currently a vacant lot, is located at the southwesterly end of Valley Centre Drive cul-de-sac in the city of San Diego, California. The site is bounded on the north by Valley Centre Drive and common driveway, on the east by Hampton Inn, on the south by Carmel Valley Road and Valley Center Court and on the west by Tio Leos' restaurant. An approximately 12 to 15-foot-high, 2:1 (horizontal to vertical) slope descends along the southern property line and about 5 to 10-foot-high, 2:1 to 4:1 slope descends along the western property line.

Our review of the ALTA/ACSM Land Title Survey for the subject site indicated building pad elevations (Lots 3 and 4) ranged from about Elevation 50 feet along the southern corner of the property to Elevation 58 feet along the northeast corner of the property adjacent to Valley Centre Drive. The building pad drains by sheet flow to the south southwest. The adjacent southerly and westerly descending slopes are covered by heavy vegetation that includes shrubs and occasional trees. No signs of slope instability were visually noted at the time of our field exploration.

Vegetation within the building pad includes very light growth of weeds and grasses. At the time of our subsurface exploration, transformers and concrete pads and storm drain outlets exist on the ground surface of the building pads.

Other existing site improvements consist of chain lined fence around the perimeter of the property building pad area, concrete walkways and underground utilities.

2.2 Background Information

Final grading on the subject property was performed around December of 1991 up to January of 1992, with observations and testing provided by Pacific Soils Engineering, Inc for the construction of the building pad. The final grade report indicated that in order to reach planned building pad elevations, fill soils were placed within the subject lots. Approximate depth of fill was about 7.5 and 26.0 feet, respectively for Lots 3 and 4. Fills consisting of sandy clay and silty sand were compacted to a minimum of 90 percent of the laboratory standard (ASTM D 1557) utilizing self-propelled, rubber tires compactors and heavy earth moving equipment.

2.3 Proposed Preliminary Project Description

Preliminary site plan provided to us indicated that the subject site will be of mixed use development that will involve a four-story medical office building, a four to five-story upscale hotel building, a single-story restaurant building, on-grade parking and one level below grade parking. Subterranean parking level will extend to depths of approximately 12 feet below existing site grade. Grading will be required to excavate for the subterranean parking. Outside the building footprint, the grade changes are anticipated to be less than one foot.

The traffic loading for the driveway and parking lot is understood to predominantly consist of automobiles with occasional heavy trucks resulting from deliveries and trash collection. Pavement designs are based on a 20-year design period. 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 5.5 for automobile drive lane areas (medium duty).

3.0 SUBSURFACE EXPLORATION

3.1 Subsurface Exploration

Our subsurface exploration was performed by representatives of our firm and consisted of the drilling of nine (9) test borings to depths of approximately 10 to 51 ½ feet below existing ground surface utilizing a hollow sterm drill 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.

Our subsurface exploration included the collection of relatively undisturbed samples of subsurface soil materials for laboratory testing purposes. Bulk samples consisted of composite soil materials obtained at selected depth intervals from the boring. Relatively undisturbed samples were collected 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 exploration logs. The central portions of the driven core samples were placed in sealed containers and transported to our laboratory for testing.

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

3.2 Subsurface Conditions

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

Soil

Our review of the *Geologic Map of San Diego Quadrangle, California* prepared by United States Geological Survey (USGS) indicated that the subject site is underlain by Very Old Paralic deposits (Unit 5) and at depths by bedrock of the Torrey Sandstone Formation (middle Eocene).

Fill and possible materials were encountered to depths of approximately 4 to 27 feet below existing grades and consisted generally of moist to damp, firm to dense silty sand. The fills were properly compacted based on the blow counts noted during our subsurface exploration. The shallower fills were located within Lot 3 and increasing in depths towards the southerly descending slope by Lot 4.

Native soils (Very Old Paralic deposits) encountered underneath the fill and possible fill generally consisted of moist, firm to dense clayey to silty fine to coarse sand and sand.

Bedrock materials of the Torrey Sandstone Formation were encountered beneath the native soils at depths of approximately 35 to 45 feet below ground surfaces and consisted of dense to very dense medium to coarse grained sandstone with some silt and clay.

Groundwater

Groundwater was encountered during our subsurface investigation to depths of about 27 to 31 feet below existing ground surfaces. However, fluctuations of the groundwater table, localized zones of perched water, and rise in soil moisture content should be anticipated during and after the rainy season. Irrigation of landscape areas on or adjacent to the site can also cause fluctuations of local or shallow perched groundwater levels.

4.0 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. No chemical analyses for environmental consideration have been conducted on the soils obtained during our subsurface exploration. The following are brief description of our laboratory test results.

In Situ Moisture and Density

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

Sieve Analysis

Sieve Analyses (Passing No. 200 Sieve) were performed on selected samples from various depths within Test Boring 1 to assist in soil classification. These tests were performed in accordance with Test Method ASTM D 1140. The results of these tests are presented in Appendix A and indicated that on-site fills and native soils are generally granular.

Expansive Potential

To evaluate the expansive potential of the near surface soils encountered during our subsurface exploration, a composite sample collected from Test Boring B-3 (1 to 5 feet) was subjected to Expansive Index (EI) testing in accordance with Test Method ASTM D 4829-08a. The result of our expansion index (EI) test indicates that the near surface sample has a low expansion potential (EI=23).

Consolidation Test

Settlement predictions under anticipated loads were made on the basis of one-dimensional consolidation test. These tests were performed in general conformance with Test Method ASTM D 4546. Loads were applied in a geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were saturated at an overburden of 1.6 ksf to evaluate the effect of a sudden increase in moisture content. Results of the consolidation tests indicated that tested on-site soils exhibit a slight degree of hydro-collapse potential (0.35%, 1.22% and 1.98%). The results of consolidation tests are graphically presented as Figures 2 and 4 in Appendix A.

Direct Shear

The angle of internal friction and cohesion was determined for a relatively undisturbed soil samples collected from Test Boring B-5. This test was performed in general accordance with Test Method No. ASTM D 3080-98. Three specimens were prepared for the test. The test specimens were artificially saturated, and then sheared under various normal loads. Results are graphically presented as Figure 5 in Appendix A.

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of the on-site fill soils were determined for a selected sample in accordance with Method A of ASTM D 1557-02. Pertinent test values are presented as Figure 6 in Appendix A.

Soluble Sulfate Analysis and Soil Corrosivity

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

Parameter	**B-1** **1 to 5 feet**
pH	7.72
Chloride	72 ppm
Sulfate	0.0105%
Resistivity	1000 ohm-cm

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

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

Corrosivity testing also included determination of the concentrations of water-soluble sulfates present in the tested soil sample. Our laboratory test data indicated that near surface soils contain approximately 0.0105 percent of water soluble sulfates. Based on Section 1904.3 of the 2013 California Building Code (CBC), concrete that may be exposed to sulfate containing soils shall comply with the provisions of ACI 318-05, Section 4.3. Therefore, according to Table 4.3.1 of the ACI 318-05, a negligible exposure to sulfate can be expected for concrete placed in contact with the 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

The project site is located in the highly seismic Southern California region within the influence of several fault systems. However, the site does not lie within the boundaries of an Earthquake Fault Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act.

5.2 Seismic Hazard Zones

Our review of the published City of San Diego Seismic Safety Study, Geologic Hazards and Faults Map (dated 4/3/2008) indicates that the subject site does not lie within a designated Liquefaction Hazard Zone. Per the City of San Diego maps, the site is located within Geologic Hazard Category 52, an area of gently sloping to steep terrain, favorable geologic structure and Low risk.

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

5.3 Landslide Hazards

The subject site does not lie within the designated Landslide Hazard Zone based on our review of the published City of San Diego Seismic Safety Study, Geologic Hazards and Faults Map. Since the subject site is underlain by compacted engineered fill, Very Old Paralic deposits and at depths by bedrock of Torrey Sandstone Formation and not located near unstable slope, mitigation of landslide hazards is not necessary for the site.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

Faulting/Seismic Design Parameters

Research of available maps published by the California Geological Survey (CGS) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The potential for fault rupture through the site is, therefore, considered to be low. The site may however be subject to strong groundshaking during seismic activity. The proposed structure should be designed in accordance with the current version of the 2013 California Building Code (CBC) and applicable local codes. Based upon the encountered subsurface soils, a Site Class C is recommended for design.

According to the maps of known active fault near-source zones (ICBO, 1998) to be used with the 2013 CBC, the Rose Canyon fault is the closest known active fault and is located approximately 4.8 kilometers to the site, with an anticipated maximum moment magnitude (Mw) of 6.9.

Within the International Code Council's 2012 International Building Code (IBC), the five-percent damped design spectral response accelerations at short periods, S_{DS}, and at 1-second period, S_{D1}, are used to determine the seismic design base shear. These parameters, which are a function of the site's seismicity and soil, are also used as parts of triggers for other code requirements. The following values are determined by using USGS Design Maps.

IBC 2012/ CBC 2013, Earthquake Loads	
Site Class Definition (Table 1613.3.2)	C
Mapped Spectral Response Acceleration Parameter, S_s (Figure 1613.3.1 (1) for 0.2 second)	1.131
Mapped Spectral Response Acceleration Parameter, S_1 (Figure 1613.3.1 (2) for 1.0 second)	0.436
Site Coefficient, F_a (Table 1613.3.3 (1) short period)	1.0
Site Coefficient, F_v (Table 1613.3.3 (2) 1-second period)	1.5
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{MS} (Eq. 16-37)	1.131
Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameter, S_{M1} (Eq. 16-38)	0.594
Design Spectral Response Acceleration Parameter, S_{DS} (Eq. 16-39)	0.754
Design Spectral Response Acceleration Parameter, S_{D1} (Eq. 16-40)	0.396

Liquefaction

Liquefaction is the loss of strength in generally cohesionless and low-plastic, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, and relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth below ground surface in which the occurrence of liquefaction may impact development and structures supported on shallow foundations is typically considered to be 50 feet. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971).

Based on the published City of San Diego Seismic Safety Study, Geologic Hazards and Faults Map (dated 4/3/2008), the Liquefaction County of San Diego Hazard Mitigation Planning prepared by URS, the depths of groundwater and dense to very dense fills, native and bedrock materials, it is our opinion that liquefaction potential for the subject site is low and not significant to the proposed development.

6.2 Site Development 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 operations for the proposed development will include the removal of any vegetation and debris within the proposed site development. All soils disturbed during site clearing should also be removed to expose suitable bearing soils.

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

It is expected that the bottoms of the excavations for the subterranean levels will expose competent materials; therefore, additional over-excavation and recompaction of these materials should not be required. However, any soils disturbed during excavation of the subterranean garage should be replaced as properly compacted fill.

Existing Utilities

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

Building Areas

Based on the results of our subsurface exploration, laboratory testing and review of the referenced soils report, the fill materials encountered within the subject site were considered to be properly compacted per local code requirements. Therefore, the bottoms of excavations for the subterranean levels will not require over-excavation and recompaction. However, any soils disturbed during excavation of the subterranean levels should be replaced as properly compacted fill.

After site stripping, building areas to remain near existing grade (restaurant building) or to receive new fill should be over-excavated to depths of about 2 feet below existing grades or planned grades and at least 1 foot below the bottom of footing, whichever is deeper, and be extended at least 3 feet beyond the building footprint. The soils exposed at the base of this recommended over-excavation should be examined by the geotechnical engineer to document that the soils are suitable for building support and additional over-excavation may be required if needed. Prior to placing structural fill, exposed bottom surfaces in each removal area approved for fill should first be scarified to a depth of at least 6 inches, moisture conditioned as necessary, and then recompacted in place to a minimum 90 percent of the maximum dry density as determined by Modified Proctor (ASTM D1557).

Removal of unsuitable materials and replacement with structural fill should be performed in accordance with the "Guide Specifications" included in the Appendix which are presented to also assist in preparing the project plans and specifications

Proofroll and Compact Subgrade

Following site clearing, the subgrades within the proposed building and pavement areas should be proofrolled in the presence of the geotechnical engineer with appropriate rubber-tire mounted heavy construction equipment or a loaded truck to detect very loose/soft yielding soil which should be removed to a stable subgrade. Following proofrolling and completion of any necessary over-excavation, the subgrade should be scarified to a minimum depth of 12 inches, moisture conditioned and recompacted to at least 90 percent of the Modified Proctor (ASTM D1557-00) maximum density. The upper 1 foot of the pavement subgrade should have minimum in-place density of at least 95% of the maximum dry density. Low areas and excavations may then be backfilled in lifts with suitable very low expansive (EI less than 21) structural compacted fill. The selection, placement and compaction of structural fill should be performed in accordance with the project specifications.

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

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

All utility trench backfill should be placed in lifts no greater than 12 to 18 inches in thickness, moisture conditioned and then compacted in place to a minimum relative compaction of 90 percent of the soil's maximum density. A representative of the project geotechnical engineer should probe and test the backfills to document adequacy of compaction.

Reuse of On-site Soil

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

Import Structural Fill

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

In addition to expansion criteria, soils imported to the site should exhibit adequate shear strength characteristics for the recommended allowable soil bearing pressure; soluble sulfate content and corrosivity; and pavement support characteristics.

Subgrade Protection

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

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

Fill Placement

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

All on-site fill should be placed in 8-inch-thick maximum loose lifts, moisture conditioned and then compacted in place to at least 90 percent of the Modified Proctor maximum density 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.

6.3 Construction Considerations

Construction Dewatering

As mentioned previously, groundwater was encountered at depths of about 27 to 31 feet below existing ground surfaces during our subsurface exploration. However, the site may be susceptible to the development of shallow perched water conditions. In the event that shallow perched water is encountered, filter sump pumps placed within pits in the bottoms of excavations are expected to be the most feasible method of construction dewatering.

Soil Excavation

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

A temporary excavation up to approximately 13 feet high will be required during construction of the subterranean parking. The excavation sidewalls are expected to expose competent fill and native soils. As observed within our borings, the on-site fill and native soils consist generally of firm to dense silty sand. Based on these conditions, temporary excavations may be cut vertical to a height of 4 feet and lay back at a maximum slope ratio of 3/4:1 (horizontal to vertical) or flatter, in order to maintain temporary stability. However, there appears to not be sufficient space to lay back the sidewalls of the temporary excavation for the recommended slope configuration without encroaching into the adjacent slope and common driveway. Therefore, shoring may be required.

Temporary Shoring

There are several shoring techniques available for the proposed excavation (i.e. soldier pile and lagging wall with or without tie-backs, sheet-pile wall with or without tie-backs). It is not known at this time what type of shoring is proposed for the subject site. However, we expect that soldier piles and wood or steel lagging would be the most likely shoring system. The temporary shoring requirements should be determined once the details for the planned retaining wall have been determined and the possibility of incorporating the temporary shoring into the final wall design.

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used, with a lateral pressure equal to that developed by a fluid with a density of 40 pounds per cubic foot for retained soils with level backfill. The project structural engineer should design the shoring system using a suitable factor of safety (i.e. minimum 1.25 factor of safety for temporary shoring).

For design of tied-back or braced shoring, we recommend the use of trapezoidal distribution of earth pressure. The maximum pressure equals to 22H in psf, where H is the height of the shoring in feet.

For the design of soldier piles (if selected), an allowable lateral bearing value (passive value) of 750 pounds per square foot per foot of depth may be assumed for soils below the level of excavation to determine soldier pile depth and spacing. However, passive resistance should be ignored within the upper foot due to possible disturbance. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils.

Minimum clear spacing between piles should be at least two effective pile diameters, sidewall to sidewall. The construction of the shoring system should be monitored continuously and adjacent structures should be observed for any potential lateral and vertical movement.

We recommend that the lagging be designed for an equivalent earth pressure of 40 pcf to a maximum value of 400 pounds per square foot. The pressure distribution for the lagging may be assumed to be semi-circular, where the pressure at the soldier piles is 0 and the pressure at the center of the lagging is 400 pounds per square foot. Additionally, the upper one foot of the lagging should be grouted or slurry–filled to assist in diverting surface water from migrating behind the shoring walls.

In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to the streets and vehicular traffic areas should be designed to resist a uniform lateral pressure of 120 pounds per square foot as a traffic surcharge factor. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

It is extremely difficult to predict accurately the amount of deflection of a shored embankment. It should be realized that some deflection will occur. We estimate that this deflection may be in the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement to the adjacent buildings or structures to remain. If it is desired to reduce the deflection of the shoring, a greater lateral earth pressure may be used in the shoring design with an increased stiffness of the system.

All soldier pile installations should be observed by the project geotechnical consultant to verify that they are cast against anticipated conditions, that the pile excavations are properly prepared and cleaned out, that the proper dimensions are achieved, and that the proper installation procedures are followed. The shoring to be constructed at the site should be surveyed and monitored for any movement. If any significant movement is observed during shoring and construction operations, it should be brought to the immediate attention of the project geotechnical consultant for appropriate corrective measures.

6.4 Foundation Recommendations

Upon completion of the recommended building pad preparation, the proposed structure may be supported by a shallow foundation system consisting of a continuous strip footing for support of the perimeter walls with a thickened slab for support of interior walls. The strip footings that will support the perimeter bearing walls footings may be designed for a maximum, net, allowable soil-bearing pressure of 3,000 pounds per square foot (psf) for square pad and continuous strip footings. Minimum footing widths are recommended to be 14 and 24 inches for walls and columns, respectively.

Foundation Setbacks and Deepened Footings

The building foundations and structures proposed near the top of the existing southerly descending slope should be supported on either deepened footings or caissons in order to mitigate the adverse effects of slope creep and to comply with the 2013 California Building Code (CBC) Section 1805.3. The 2013 CBC states that a horizontal structural setback equivalent to one-third the total slope height (to a maximum of 40 horizontal feet) is required between the outside bottom edges of the proposed footings and the face of the adjacent descending slope. The subject site adjacent southerly slope and portion of the south easterly slope has approximately 12 to 15 feet in height; therefore, a structural setback of 5 feet (outside bottom edge of the footing to the face of the adjacent descending slope) is required to meet the 2013 CBC Section 1805.3.

Reinforcing

The minimum steel reinforcing within continuous wall footings is recommended to consist of at least four No. 5 bars (2 top and 2 bottom) continuous through any intermittent column footings. The recommended reinforcing pertains to a minimum 12-inch thick, maximum 24-inch wide footing pad; additional steel reinforcing may be needed if a thinner or wider footing pad is used to provide equivalent rigidity. The recommended quantity of reinforcing is intended to provide some increased footing rigidity and allow the footings to span greater distances. Conventional steel reinforcing may be used in column pad footings. The design of the foundations as well as determination of the actual quantity of steel reinforcing and the footing dimensions should be performed by the structural engineer.

Lateral Load Resistance

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

A coefficient of friction of 0.35 may be used with dead load forces for footings placed on newly placed compacted fill soil. An allowable passive earth pressure of 250 psf per foot of footing depth (pcf) below the lowest adjacent grade may be used for the sides of footings placed against newly placed structural fill.

Bearing Material Criteria

Structural fill placed and compacted under engineering controlled conditions continuous from a suitable existing fill and native soils are considered to be suitable for direct foundation support. Soil suitable to serve as the subgrade for placement of structural fill within the zone of footing influence should exhibit at least a firm relative density (average N value of at least 10) for non-cohesive soils for the recommended allowable soil bearing pressure. For design and construction estimating purposes, suitable bearing soils are expected to be encountered at nominal foundation depths, following site grading activities.

Evaluation of the subgrade within the zone of footing influence prior to fill placement should be performed using appropriate bearing capacity testing methods and in-situ testing equipment such as dynamic or static cone penetrometers depending upon the material and should typically include testing to a depth of 3 feet below the structural fill subgrade. The actual depth of evaluation may be revised at the discretion of 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.

Foundation Embedment

As mentioned previously, the exterior foundations of the new building and/or basement footings adjacent to the descending southerly slopes should have a structural setback of at least 5 feet to comply with 2013 CBS Section 1805.3. However, exterior foundations located at least 5 feet away from the top of the slope should extend at least 18 inches below the adjacent exterior grade for bearing capacity consideration. Interior footings may be supported at nominal depth below the floor. All footings must be protected against weather and water damage during and after construction, and must be supported within suitable bearing materials.

Estimated Foundation Movement

Post-construction total and differential settlement of a shallow foundation system designed and constructed in accordance with the recommendations provided in this report are estimated to be less than ¾ and ½ inch, respectively, 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 structures provided it is considered in the structural design.

6.5 On-Grade Floor Slab Recommendations

Subgrade

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

Design

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

The floor slab is recommended to be underlain by a 4 inch thick layer of granular material. A minimum 10-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). It is recommended that a structural engineer or architect specify the vapor retarder location with careful consideration of concrete curing and the effects of moisture on future flooring materials. The vapor retarder is recommended to be in accordance with ASTM E 1745-97, which is entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. The sheets of the vapor retarder material should be evaluated for holes and/or punctures prior to placement and the edges overlapped and taped. If materials underlying the synthetic sheet contain sharp, angular particles, a layer of coarse sand (Sand Equivalent>30) approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of coarse sand may be needed between the slab and the vapor retarder to promote proper curing. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

Estimated Movements

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

6.6 Basement Floor Slab (Below Grade Parking)

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

Design

The ground floor basement, below-grade parking of the proposed structure may be designed and constructed as a slab-on-grade supported by a properly prepared subgrade. The floor slab is recommended to be a minimum of 5 inches in thickness to provide adequate cover for the recommended reinforcing. Due to the potential variability of the subgrade and concentrated loading by moving and parked vehicles, it is recommended to consist of #3 rebars placed at 18 inches on-center, each way and placed at mid-height in the slab. It is recommended that a structural engineer or architect specify the floor slab thickness, reinforcing, joint details and other parameters.

A minimum 10-mil synthetic sheet should be placed below the floor slab to serve as a vapor retarder where required to protect moisture sensitive floor coverings (i.e. tile, or carpet, etc.). 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 sand approximately 2 inches thick or a geotextile should be provided to protect it from puncture. An additional 2-inch thick layer of sand may be needed between the slab and the vapor barrier to promote proper curing. The sand layers above and below the synthetic sheeting may be used as a substitute for the granular material below the slab. Proper curing techniques are recommended to reduce the potential for shrinkage cracking and slab curling.

A minimum 6 inch thick aggregate base course should be beneath the vapor retarder below-grade floor slabs to serve as a capillary break and drainage layer. A non-woven geotextile consisting of Mirafi 140N or other non-woven geotextile selected for its separation and filtration properties and approved by Giles prior to placement, is recommended to be placed on the prepared basement floor subgrade. The non-woven geotextile is intended to reduce migration of the fine material into the drainage course and drainage system. The base course should consist of free-draining materials.

The base course below the below-grade floor slab is recommended to contain minimum 3-inch diameter perforated lateral drain pipes to provide increased drainage below the basement floor. The lateral drain pipes should be continuous across the building length or width and have a minimum spacing of 20 feet. Increased below floor drainage system capacity is recommended due to the shallow groundwater table at the site and the potential for fluctuation of the groundwater conditions.

Estimated Movement

With proper site preparation and construction observation, the post-construction total and differential settlement are estimated to be less than ½ and ¼ -inch, respectively. The estimated differential movement is anticipated to occur across a span equivalent to one-half of the short dimension of the structure and is expected to be within tolerable levels provided it is considered in the structural design.

6.7 Retaining Wall Recommendations

The project includes walls below grade for the subterranean parking level and may also include shallow retaining walls supporting soil materials such as the ramp areas accessing the below-grade parking. Retaining walls for the basement levels can be founded on shallow foundations in accordance with the recommendations presented in this report. Design lateral earth pressure, backfill criteria, and drainage recommendations for walls below grade are presented below.

Static Lateral Earth Pressures

Retaining walls should be designed to resist the applicable lateral earth pressures. On-site soil materials may be used as backfill behind the retaining walls provided they are confirmed to have very low to low expansive characteristic and allow for a drainage layer as discussed in subsequent paragraphs. For on-site soils and/or imported soils (EI less than 21) to be used as backfill materials, an active earth pressure of 40 pounds per cubic foot (equivalent fluid pressure) should be used assuming a level adjacent backfill and drained conditions. For walls to be restrained at the top, an at-rest pressure of 60 pcf should be used for design. All retaining walls should be supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings and vehicles in addition to the above recommended active earth pressure.

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

Drainage and Damp-proofing

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

Wall backfill should be protected against infiltration of surface water. Backfill adjacent to walls should be sloped so that surface water drains freely away from the wall and will not pond. Damp-proofing of walls below-grade is recommended especially where moisture control is required by an approved waterproofing compound or covered with similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

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

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

Seismic Pressure

Per 2013 CBC Section 1803.5.12, retaining walls supporting more than 6 feet of backfill height shall be designed to resist the additional earth pressure caused by seismic ground shaking. The planned retaining wall for the site will be greater than 6 feet high and therefore, the seismic lateral pressure must be added to the static lateral pressure distribution for the design of the walls. A seismic lateral pressure of 15 pounds per cubic foot (pcf) is recommended for the subject site using the simplified Mononobe-Okabe formula. The resultant seismic lateral force acts at 0.6H above the base of the wall using an inverted triangular pressure distribution. The overturning resultant should also fall within the center third (kern) of the retaining wall footing for stability, or the design must be re-evaluated with a reduced bearing area. Where earthquake loads are included in the design, the minimum safety factor for retaining wall sliding and overturning shall be 1.1 (Section 1807.2.3).

6.8 Pavement Recommendations

Pavement Subgrades

Following completion of the recommended subgrade preparation procedures, the subgrade in areas of new pavement construction are expected to consist of existing on-site soils that exhibit a very low to low expansion potential. The anticipated subgrade soils are classified as a good subgrade material with estimated R-value of 20 to 30 when properly prepared based on the Unified Soil Classification System designation of SM/SC. An R-value of 20 has been assumed in the preparation of the pavement design. It should however, be recognized that the City of San Diego 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=5.5)**	
Asphaltic Concrete Surface Course (b)	1	1	Section 39, (a)
Asphaltic Concrete Binder Course (b)	2	2	Section 39, (a)
Crushed Aggregate Base Course	5	7	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. We recommend that the geotechnical engineer monitors and tests subgrade preparation, and that the subgrade be evaluated immediately before pavement construction.

Portland Concrete Pavements

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

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

General Considerations

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

6.9 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.

6.10 Basis of Report

This report is based on Giles' proposal, which is dated March 7, 2014 and is referenced by Giles' proposal number 2GP-1403002. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

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

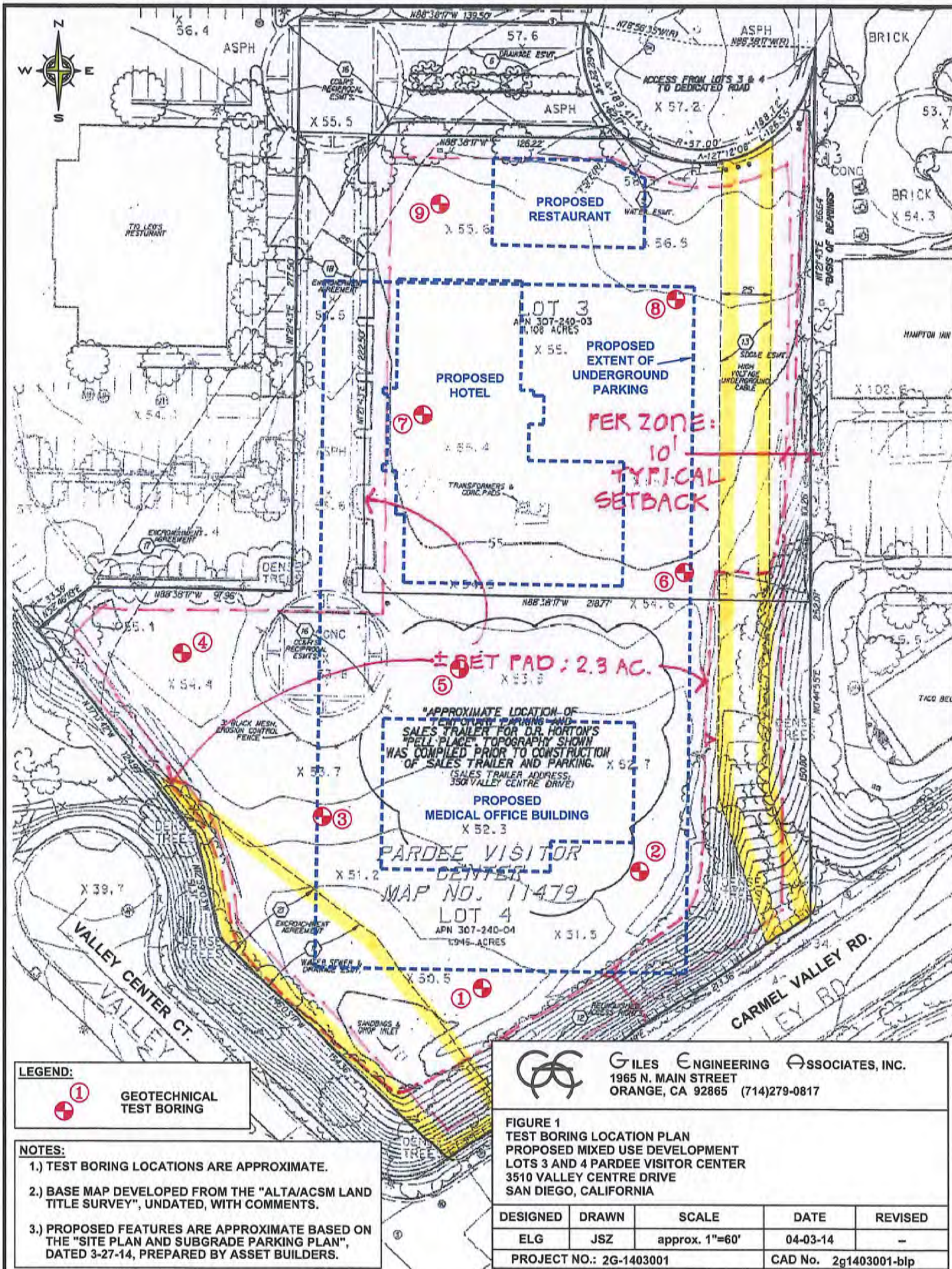
The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the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

APPENDIX A

FIGURES AND TEST BORING LOGS

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

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



LEGEND:

① GEOTECHNICAL TEST BORING

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "ALTA/ACSM LAND TITLE SURVEY", UNDATED, WITH COMMENTS.

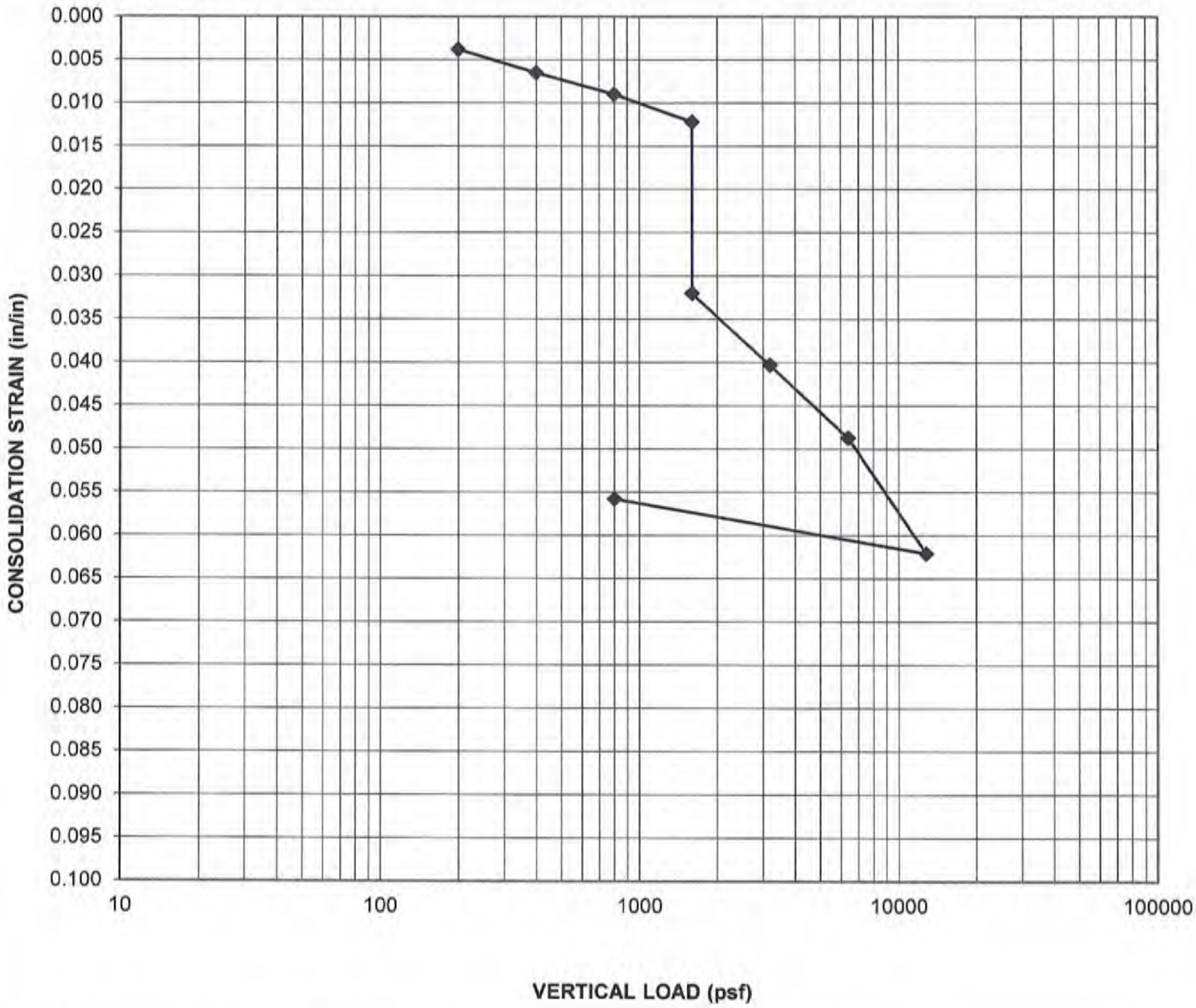
3.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "SITE PLAN AND SUBGRADE PARKING PLAN", DATED 3-27-14, PREPARED BY ASSET BUILDERS.

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

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED MIXED USE DEVELOPMENT
LOTS 3 AND 4 PARDEE VISITOR CENTER
3510 VALLEY CENTRE DRIVE
SAN DIEGO, CALIFORNIA

DESIGNED	DRAWN	SCALE	DATE	REVISED
ELG	JSZ	approx. 1"=60'	04-03-14	–
PROJECT NO.: 2G-1403001			CAD No. 2g1403001-blp	

CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty fineSand		
Boring No.	B-2		
Sample No.	4-CS	Initial Moisture Content (%)	5.9
Depth (ft.)	10.0 - 11.5	Final Moisture Content (%)	20.2
Elevation		Natural Density (pcf)	105.5
Liquid Limit	NP	Initial Dry Density (pcf)	99.6
Plastic Limit	NP	Final Dry Density (pcf)	105.4
Specimen Diameter (in.)	2.42	Collapse @ 1600 psf	1.98%
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 1600 psf pressure

Project:	Multi level building
Client:	Carmel Valley Centre Dr.
Project No.:	2G-1403001
Figure No.:	2

GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-

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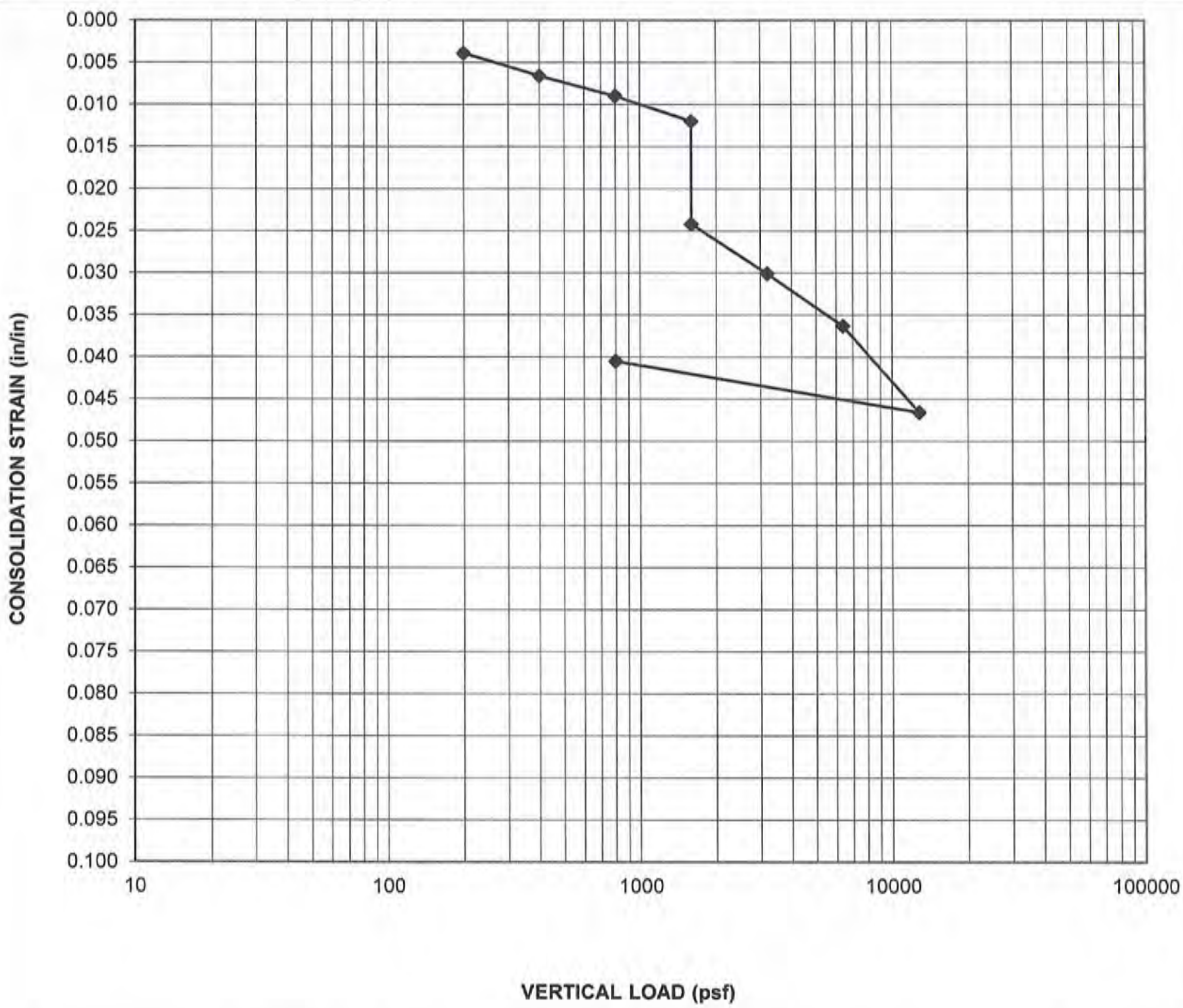
CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty fineSand		
Boring No.	B-5		
Sample No.	3-CS	Initial Moisture Content (%)	4.3
Depth (ft.)	8.5 - 10.0	Final Moisture Content (%)	20.3
Elevation		Natural Density (pcf)	105.6
Liquid Limit	NP	Initial Dry Density (pcf)	101.2
Plastic Limit	NP	Final Dry Density (pcf)	105.4
Specimen Diameter (in.)	2.42	Collapse @ 1600 psf	1.22%
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 1600 psf pressure

Project:	Multi level building
Client:	Carmel Valley Centre Dr.
Project No.:	2G-1403001
Figure No.:	3

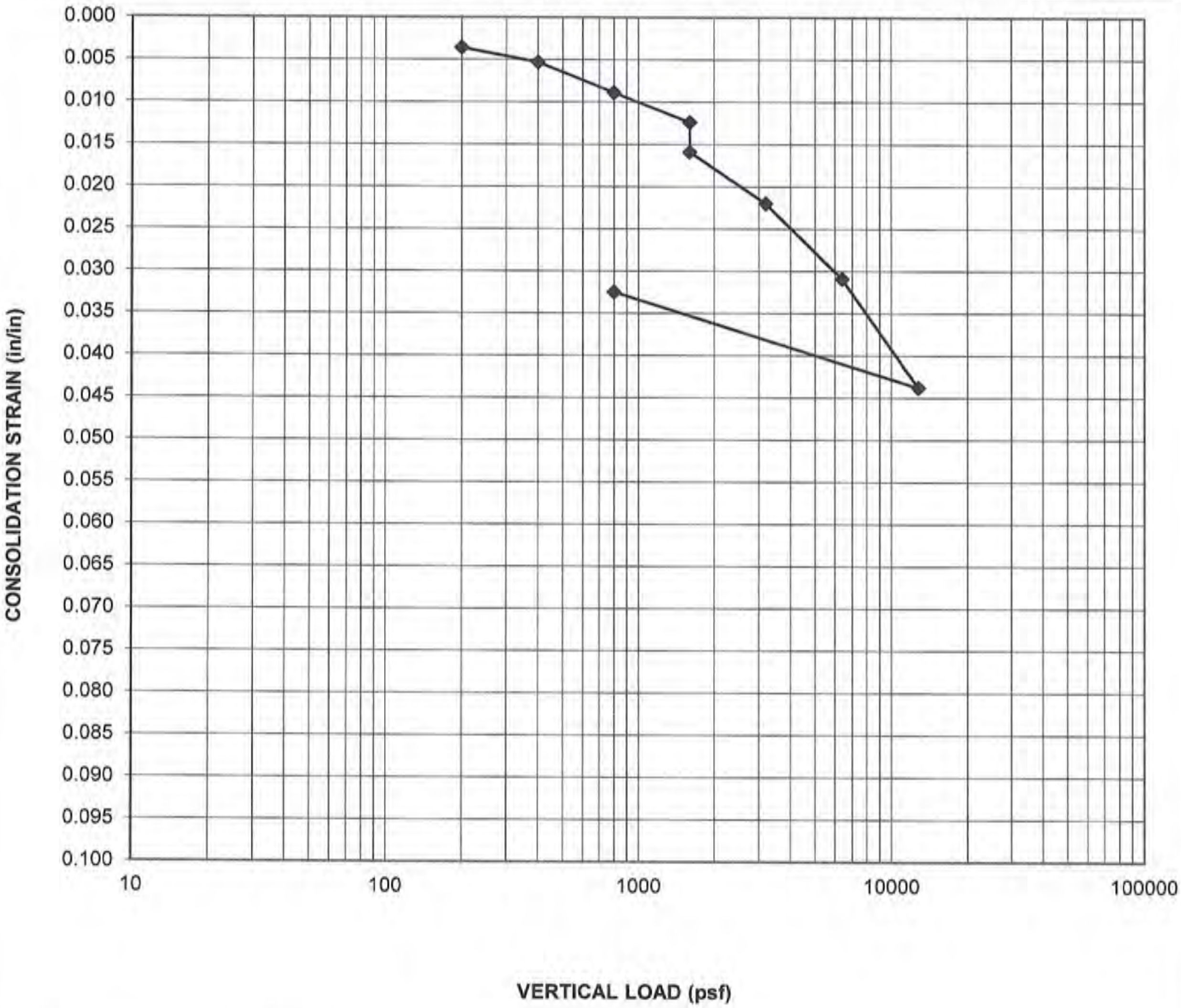
GILES ENGINEERING ASSOCIATES, INC.

-GEOTECHNICAL, ENVIRONMENTAL, AND CONSTRUCTION MATERIALS-

1965 NORTH MAIN STREET, ORANGE, CALIFORNIA

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

CONSOLIDATION / COLLAPSE TEST ASTM D2435/ASTM D5333



Classification	Silty fineSand		
Boring No.	B-5		
Sample No.	5-CS	Initial Moisture Content (%)	16.2
Depth (ft.)	18.5 - 20.0	Final Moisture Content (%)	19.3
Elevation		Natural Density (pcf)	130.9
Liquid Limit	NP	Initial Dry Density (pcf)	112.6
Plastic Limit	NP	Final Dry Density (pcf)	116.4
Specimen Diameter (in.)	2.42	Collapse @ 1600 psf	0.35%
Initial Specimen Thickness (in.)	1.00		

Sample inundated at 1600 psf pressure

Project:	Multi level building
Client:	Carmel Valley Centre Dr.
Project No.:	2G-1403001
Figure No.:	4

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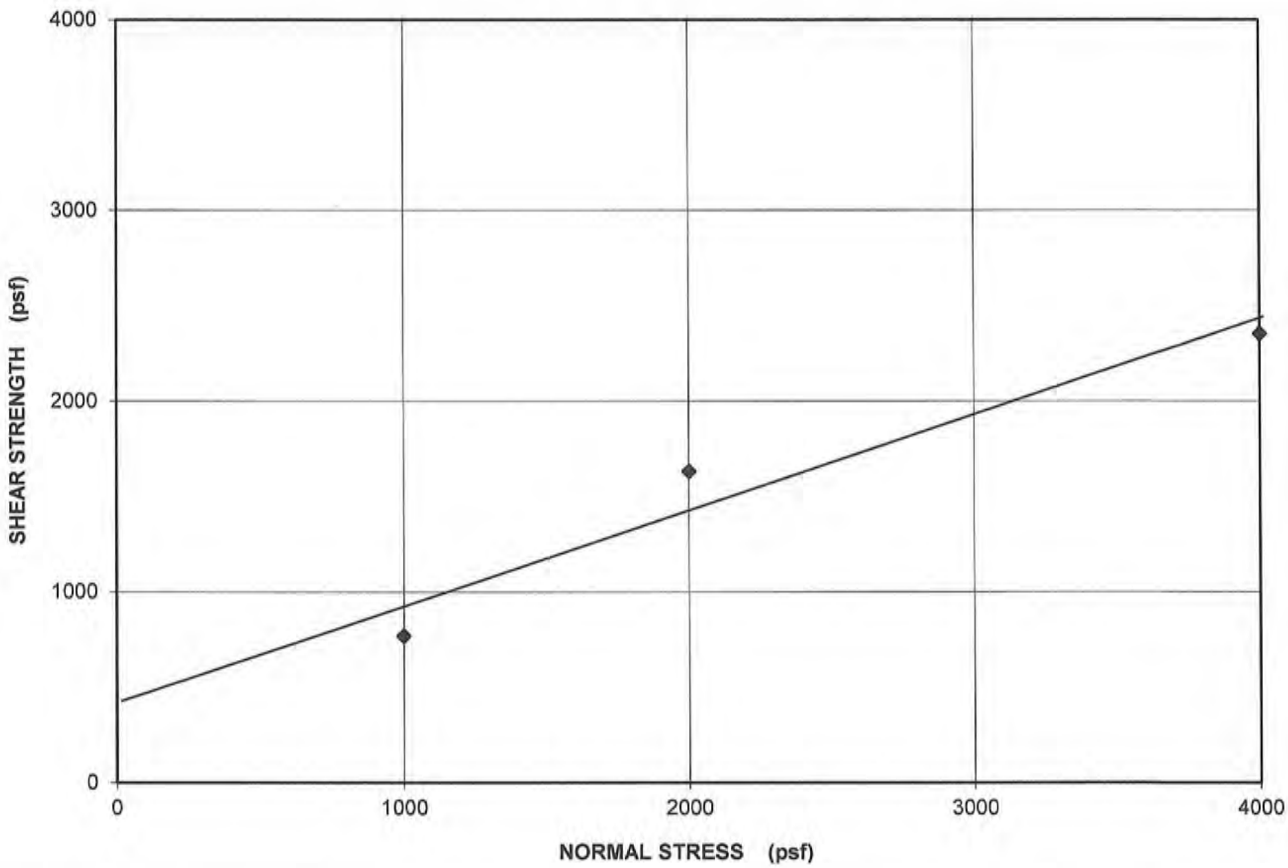
Giles Engineering Associates, Inc.

GEOTECHNICAL, ENVIRONMENTAL AND CONSTRUCTION MATERIALS CONSULTANTS

1965 N. MAIN STREET, ORANGE, CA 92865

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

UNDISTURBED DIRECT SHEAR TEST (ASTM D3080)



Classification	Sandy Clay		
Boring No.	B-5	Depth	13.5 feet

		Initial Specimen Properties:	
		Diameter (in.)	2.4
		Height (in.)	1.0
PROJECT:	Multi-level Building	Moisture Content (%)	4.2
		Remolded Density (pcf)	
CLIENT:	Carmel Valley Centre Dr. LLC	Dry Density (pcf)	114.6
		LL	
PROJECT NO.:	2G-1403001	PL	
		Ultimate - C (psi/psf)	400
FIGURE NO.	5	Ultimate - PHI (degrees)	27

Job No. 2G-1403001 Date 4/3/14

Project Proposed Mixed-Use Development - 3510 Valley Centre Drive San Diego, CA

Source of Material B-1 1.0

Description of Material Silty fine to coarse Sand

Test Method 1557A

TEST RESULTS

Maximum Dry Density 131.0 PCF

Optimun Water Content 7.5 %

ATTERBERG LIMITS

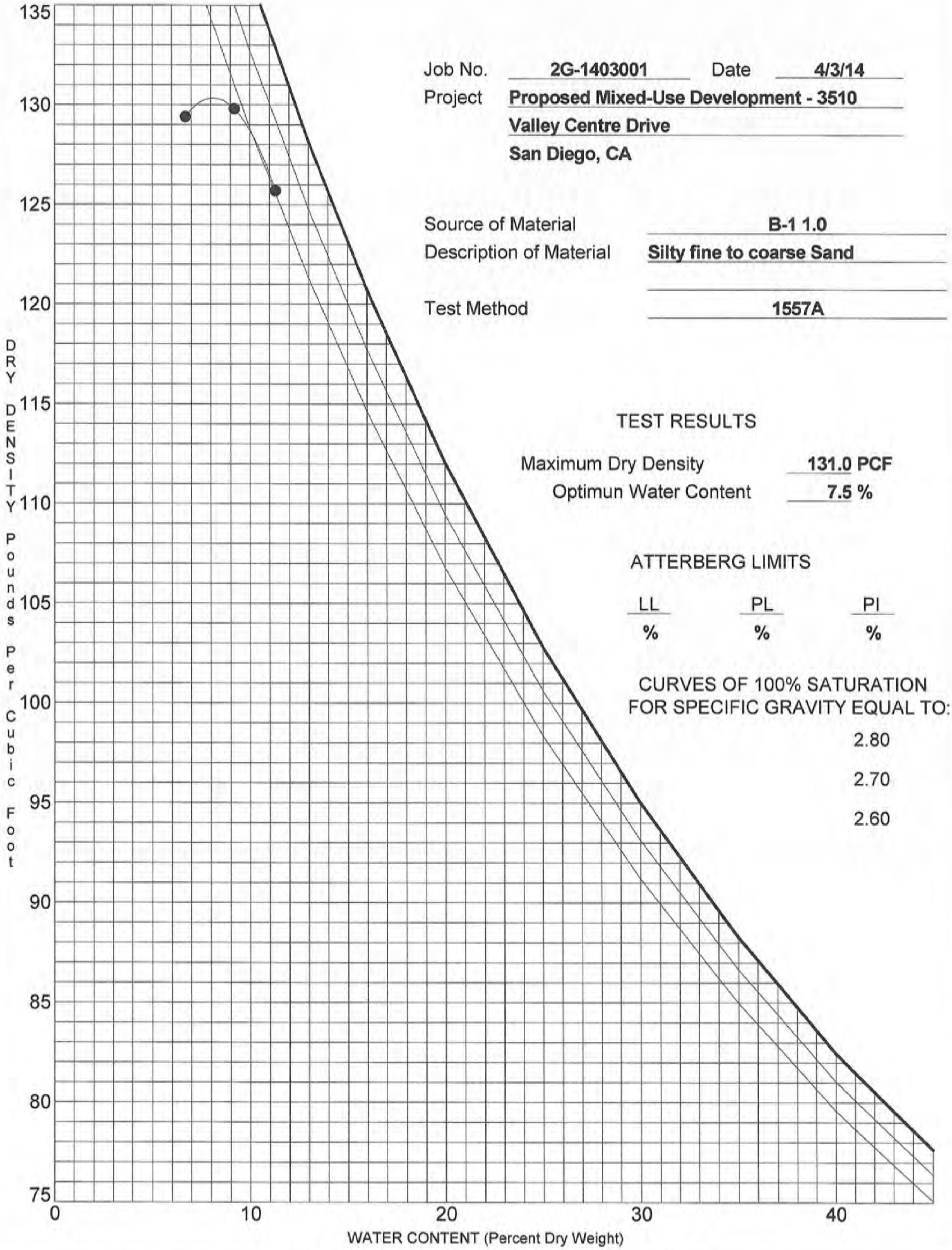
LL	PL	PI
%	%	%

CURVES OF 100% SATURATION FOR SPECIFIC GRAVITY EQUAL TO:

2.80

2.70

2.60



MOISTURE-DENSITY RELATIONSHIP

Giles Engineering Associates, Inc

FIGURE 6

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-1	PROJECT: Proposed Mixed-Use Development	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: ~50.5'	PROJECT LOCATION: 3510 Valley Centre Drive	Milwaukee Atlanta
COMPLETION DATE: 3/19/14	San Diego, CA	Dallas Wasington, D.C.
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001	Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Grayish Brown Silty fine to medium Sand - Dry to Moist (Fill)		1 SS	32				6		
Light Yellowish Brown Silty fine Sand - Damp to Moist (Fill)	5	2 SS	18				6		P_{200} = 37%
	10	3 SS	19				4		
White Silty fine Sand - Damp (Possible Fill)	15	4 SS	20				3		P_{200} = 15%
White to Yellowish Brown fine Sand - Damp to Moist (Possible Fill)	20	5 SS	34				4		
	25	6 SS	32				11		P_{200} = 21%
Light Gray fine Sandy Clay to Light Gray Clay - Wet (Native)	30	7 SS	16		1.75		27		LL = 36 PL = 22 (PI = 14)
Gray Clayey fine Sand - Moist	35	8 SS	18		4.25		13		P_{200} = 37%
	40	9 SS	22		2.75		16		P_{200} = 33%
White fine to medium Sandstone, some Clay, some Iron Oxide, slightly Micaceous - Moist (Possible Bedrock)	45	10 SS	44				16		
	50	11 SS	44		2.0		18		P_{200} = 35%

Boring terminated at 51.5 feet.
Groundwater encountered at 27 feet.

WATER OBSERVATION DATA	REMARKS
∇ WATER ENCOUNTERED DURING DRILLING: 27	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-2	PROJECT: Proposed Mixed-Use Development	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: ~51.5'	PROJECT LOCATION: 3510 Valley Centre Drive	Milwaukee Atlanta
COMPLETION DATE: 3/19/14	San Diego, CA	Dallas Wasington, D.C.
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001	Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Grayish Brown Silty fine to medium Sand - Moist (Fill)		1 SS	24				8		
Light Gray very fine Sand, some Silt, trace of Clay - Moist (Fill)		2 SS	19				13		
Light Brown Silty fine to medium Sand - Moist (Fill)	5	3 SS	29				8		Dd = 116.4 pcf
Yellowish Brown Silty fine Sand - Damp (Possible Fill)	10	4 SS	16				4		Dd = 101.4 pcf
White fine Sand, trace to little Silt - Damp (Possible Native)	15	5 SS	46				2		Dd = 103.9 pcf
	20	6 SS	46				3		Dd = 99.4 pcf

Boring terminated at 21.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-3	PROJECT: Proposed Mixed-Use Development	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: ~52.7'	PROJECT LOCATION: 3510 Valley Centre Drive	Milwaukee Atlanta
COMPLETION DATE: 3/19/14	San Diego, CA	Dallas Wasington, D.C.
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001	Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Grayish Brown Silty fine to medium Sand, some Clay - Moist (Fill)		1 SS	21				8		EI = 23 (Low)
Brownish Gray Silty fine to medium Sand - Moist (Fill)		2 SS	20				8		
	5								
		3 SS	24				10		
	10								
Light Gray Yellowish Brown Silty fine Sand, some Iron Oxide Staining - Moist (Possible Native)		4 SS	33				21		
	15								
		5 SS	19				17		

Boring terminated at 17.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
▽ WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
▼ WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION:	PROJECT:
B-4	Proposed Mixed-Use Development
SURFACE ELEVATION: ~54.4'	PROJECT LOCATION: 3510 Valley Centre Drive
COMPLETION DATE: 3/19/14	San Diego, CA
FIELD REPRESENTATIVE: A. Ceertantes	GILES PROJECT NUMBER: 2G-1403001

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Brown Silty fine to medium Sand - Moist (Fill)		1 SS	22				8		
Light Brown Silty fine to medium Sand - Moist (Fill)		2 SS	14				9		
	5	3 SS	16				11		
	10								
Brown Gray Silty fine to medium Sand, little Gravel, some Clay - Moist (Possible Fill)		4 SS	12				12		
	15								
Yellowish Brown Silty fine Sand - Moist (Possible Native)		5 SS	16				12		

Boring terminated at 17.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-5	PROJECT: Proposed Mixed-Use Development
SURFACE ELEVATION: ~53.5'	PROJECT LOCATION: 3510 Valley Centre Drive
COMPLETION DATE: 3/20/14	San Diego, CA
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Brown Silty fine Sand - Damp to Moist (Fill)		1 SS	18				7		
Yellowish Brown to Light Gray Silty fine Sand - Damp (Possible Fill)	5	2 CS	47				4		Dd = 101.6 pcf
Light Gray to Light Yellowish Brown very fine Sand, little to trace of Silt - Damp (Possible Native)	10	3 CS	58				3		Dd = 108.9 pcf
Light Gray to Yellowish Brown fine Sand, trace of Silt and Clay - Damp (Native)	15	4 CS	43				4		Dd = 114.6 pcf
Light Gray to Yellowish Brown fine Sandy Clay - Moist	20	5 CS	46				11		Dd = 104.6 pcf

Boring terminated at 20 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-6	PROJECT: Proposed Mixed-Use Development
SURFACE ELEVATION: ~54.6'	PROJECT LOCATION: 3510 Valley Centre Drive
COMPLETION DATE: 3/20/14	San Diego, CA
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	W (%)	PID	NOTES
Light Brown to Light Gray Silty fine Sand - Moist (Fill)		1 SS	22				9		
		2 CS	24				3		Dd = 107.4 pcf
	5								
Light Yellowish Brown to Light Gray Silty fine Sand - Moist (Possible Native)		3 CS	40				2		Dd = 105.6 pcf
	10								
White to Light Yellowish Brown fine Sand, little Silt - Dry to Damp (Native)		4 CS	32				2		Dd = 100.7 pcf
	15								
		5 SS	14				4		
	20								
Light Gray to Yellowish Brown Clayey fine Sand - Moist		6 CS	26				17		Dd = 102.7 pcf
	25								
		7 CS	25				17		Dd = 109.9 pcf
	30								
Light Gray fine Sandy Clay - Very Moist		8 CS	13				31		
	35								

Boring terminated at 35 feet.
Groundwater encountered at 31 feet.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: 31	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION:	PROJECT:
B-7	Proposed Mixed-Use Development
SURFACE ELEVATION:	PROJECT LOCATION:
~55.4'	3510 Valley Centre Drive
COMPLETION DATE:	
3/20/14	San Diego, CA
FIELD REPRESENTATIVE:	
A. Cervantes	GILES PROJECT NUMBER: 2G-1403001

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Brown Silty fine Sand - Moist (Fill)		1 SS	20				10		
		2 SS	20				12		
Light Yellowish Brown very fine Sand, little Silt, trace of Clay - Moist (Possible Native)	5	3 SS	21				14		
	10								
Light Gray Clay, some pockets of White fine Sand, little Silt - Very Moist (Native)		4 SS	14		4.25		31		
	15	5 SS	15		4.5		29		

Boring terminated at 17.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-8	PROJECT: Proposed Mixed-Use Development
SURFACE ELEVATION: ~56.0'	PROJECT LOCATION: 3510 Valley Centre Drive
COMPLETION DATE: 3/20/14	San Diego, CA
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001

GILES ENGINEERING ASSOCIATES, INC.
Milwaukee Atlanta
Dallas Wasington, D.C.
Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Gray to Dark Yellowish Brown fine Sandy Clay - Moist (Fill)		1 SS	13		4.5		31		EI = 52 (Medium)
Gray to Yellowish Brown fine Sandy Clay, some Silt - Moist (Possible Native)		2 CS	30		4.5+		21		Dd = 101.5 pcf
	5	3 CS	24		4.5+		23		Dd = 100.8 pcf
Gray and some Yellowish Brown Silty Clay, some fine Sand - Moist to Very Moist (Native)	10	4 CS	27		4.5+		16		Dd = 107.1 pcf
	15	5 CS	23		4.5+		27		Dd = 93.3 pcf
White to Light Gray very fine Sand, trace to little Silt - Damp to Moist	20	6 SS	26				4		
	25	7 SS	27				11		
	∇								
Light Gray fine Sand, trace of Clay, some pockets of Clay - Very Moist	30	8 SS	10				20		
Dark Yellowish Brown fine to coarse Sandstone, little fine Gravel, some Iron Oxide Staining - Moist (Possible Bedrock)	35	9 SS	50/4"				18		
	40	10 SS	50/6"				16		

Boring terminated at 41.5 feet.
Groundwater encountered at 29 feet.

WATER OBSERVATION DATA	REMARKS
∇ WATER ENCOUNTERED DURING DRILLING: 29	CS = California Split Spoon
WATER LEVEL AFTER REMOVAL:	SS = Standard Penetration Test
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: B-9	PROJECT: Proposed Mixed-Use Development	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: ~55.6'	PROJECT LOCATION: 3510 Valley Centre Drive	Milwaukee Atlanta
COMPLETION DATE: 3/20/14	San Diego, CA	Dallas Wasington, D.C.
FIELD REPRESENTATIVE: A. Cervantes	GILES PROJECT NUMBER: 2G-1403001	Los Angeles Orlando

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
Light Brown Silty fine Sand - Moist		1 SS	21				9		
Light Brown to Light Gray fine Sand, little Silt - Moist (Possible Native)		2 SS	20				9		
	5	3 SS	43				7		
	10								
Light Gray to Light Yellowish Brown very fine Sand, little Silt - Damp (Native)		4 SS	41				4		
	15								
Light Yellowish Brown Silty fine Sand - Moist		5 SS	20				18		

Boring terminated at 17.5 feet.
No groundwater encountered.

WATER OBSERVATION DATA	REMARKS
WATER ENCOUNTERED DURING DRILLING: None	SS = Standard Penetration Test
WATER LEVEL AFTER REMOVAL:	
CAVE DEPTH AFTER REMOVAL:	
WATER LEVEL AFTER HOURS:	
CAVE DEPTH AFTER HOURS:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between 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.

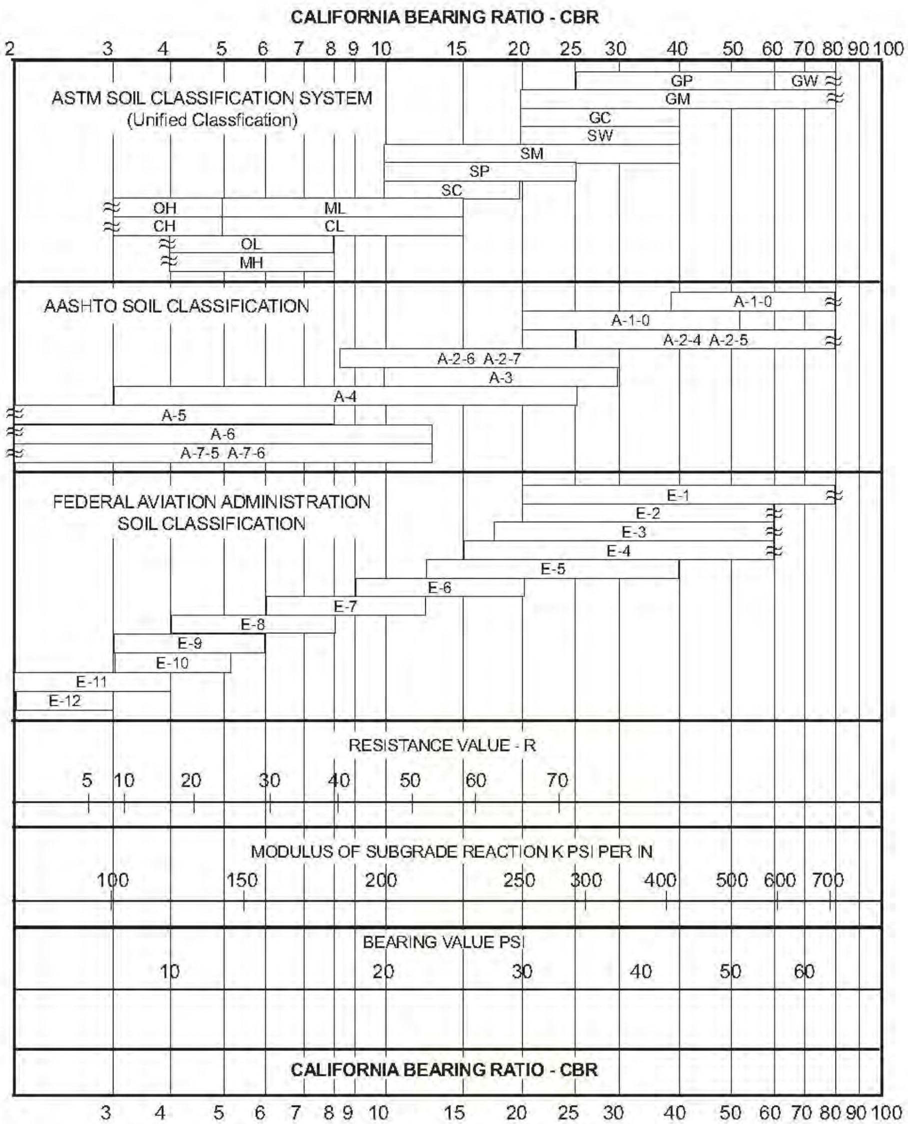
Laboratory Testing

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

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.

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.

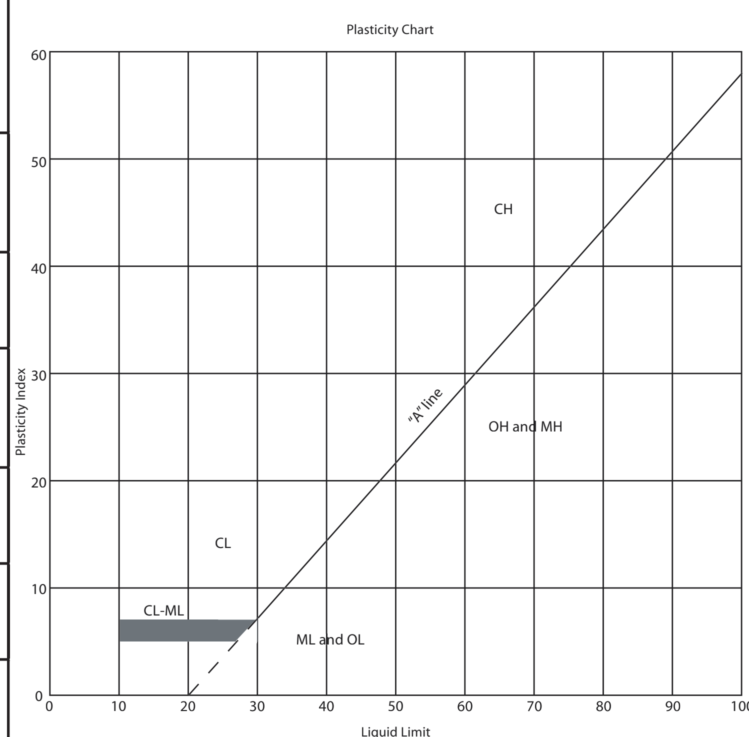
8. Wherever, in the opinion of the soils engineer or the Owner's Representatives, an unstable condition is being created either by cutting or filling, the work should not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised,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

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 / u	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
			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 / u	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
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt	Peat and other highly organic soils			

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

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

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DESCRIPTIVE TERM (% BY DRY WEIGHT)

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

PARTICLE SIZE (DIAMETER)

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

SOIL PROPERTY SYMBOLS

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

DRILLING AND SAMPLING SYMBOLS

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

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

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

NON-COHESIVE (GRANULAR) SOILS

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

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

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

Important Information About Your
Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

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