FINAL REPORT
GEOTECHNICAL STUDY

PROPOSED HOTEL

550 GATEWAY BOULEVARD
SOUTH SAN FRANCISCO, CALIFORNIA

Prepared for:
SOUTHERN HOSPITALITY SERVICES, LLC

Prepared by:
GEC

February 2016

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Project No.: P16-122
February 12, 2016

Mr. VJ Patel
Southern Hospitality Services, Llc
Holiday Inn Express
2834 El Camino Real
Redwood City, California 94061

Subject: Proposed Hotel
550 Gateway Boulevard
South San Francisco, CA 94024
Geotechnical Investigation Report

Dear Mr. Patel:

In accordance with your authorization, GeoEngineering Consultants (GEC) has performed a geotechnical investigation at the subject site located in South San Francisco, California.

This report summarizes our findings, conclusions and recommendations for use in consideration of proposed hotel at the above-referenced address, based on the subsurface investigation performed to date.

1. PURPOSE AND SCOPE

The purpose of the investigation was to determine the surface and subsurface soil conditions at the proposed hotel located in South San Francisco, California. Based on the results of our investigation, recommendations are provided for construction of the proposed hotel at the afore-mentioned site.

Our investigation included the following:

a. Field reconnaissance by the Soil Engineer;

b. Evaluation of the general geology and seismicity of the site;

c. Drilling and sampling of the subsurface soils;

d. Geotechnical laboratory testing;

e. Analysis of the data and formulation of conclusions and recommendations, and

f. Preparation of this written report.

Details of our field investigation are presented in Appendix A.

2. PROJECT DESCRIPTION

Based on our site visit and our discussions with you, the proposed project is understood to consist of constructing one L-shaped 4 to 5 stories wood-framed hotel structure without basement at the subject site. Footing loads are anticipated to be moderate. On-site at grade parking and driveway areas and landscaping are also planned for the development of the project.

3. SITE DESCRIPTION

The entire site is one parcel (APN: 015023270) approximately 87,200 square feet in plan view. The proposed site is bounded by Gateway Boulevard to the northwest and other commercial buildings to the other directions. Three PG&E high voltage power extend to the southwest. There are several trees at the site along Gateway Boulevard and . Currently, the site is vacant. A landscaped berm, approximately 5 feet high, runs along the northwest side of the property adjacent to Gateway Boulevard, and then diagonally across the northern corner of the site. Several large trees and ground cover were present along the full length of the berm. The remainder of the site was generally flat, with a 3 foot difference in elevation trending north to south.

Topographically, the entire site is located on a flat ground. Drainage appears to follow the local topography from south toward the north along the local topography. The approximate location of the site is shown on Plate 1, "Site Vicinity Map"' in Appendix A.

The site location and description is based on a site reconnaissance by the Soil Engineer.

4. REGIONAL GEOLOGIC SETTING

The subject site is located within the Coast Ranges geomorphic province and consists of a belt of sedimentary, volcanic, and metamorphic rocks, which extend from southern California to Oregon. The structural geology of the Coast Ranges is complex and dominated by transpressive stress (combined transform and compressional) concentrated along faults within the San Andreas Fault system. On the eastern portion of the San Francisco Bay, bedrock geology consists of sedimentary and metamorphic rocks ranging from Cretaceous through Quaternary periods (up to 144 million years to present).

The subject site is located east of the Santa Cruz Mountains. A large portion of the City of South San Francisco, primarily east of U.S. 101, is underlain by deposits of Bay Mud up to 80 feet deep in some places. This site is mapped as being underlain by Early to late Pleistocene undifferentiated alluvial deposits (Qoa) by Knudsen and others, (2000). Deposits mapped within Qoa can include alluvial fan, stream terrace, basin and channel deposits. Qoa includes the Colma Formation on the San Francisco Peninsula (Bonilla, 1971; Schlocker, 1974), which has been described as a marine, estuarine and fluvial, unconsolidated fine to medium sand with silt and clay.

The subject site is not located within any state or local hazard zone with respect to fault rupture, landsliding, compressible soils, or dike failure.

5. FAULTS AND SEISMICITY

South San Francisco is located in one of the most seismically active regions in the United States. No known active faults traverse the City of South San Francisco and no Alquist-Priolo Earthquake Fault Zoning has been established by the state. However, the city is located between the active San Andreas and Hayward faults, as well as numerous smaller faults.

The principal active faults in the vicinity are the San Andreas, about 4.7 kilometers to the southwest, the San Gregorio about 15 kilometers to the southwest, the Monte Vista-Shannon about21 kilometers to the southwest, the Hayward about 27 kilometers to the east, and the Calaveras about 41 kilometers to the east. The Peninsula segment of the San Andreas Fault, the predominant fault system in California, passes through the westernmost corner of South San Francisco, commonly referred to as the Westborough area.

The site is not within one of the Alquist-Priolo Earthquake Fault Zones established by the CGS around known active faults. The closest known active fault is the San Andreas which situated approximately 4.7 kilometers southwest of the site.

The San Francisco Bay Area has experienced several large earthquakes during historical times. The following paragraphs present Richter magnitudes of notable bay-area historic earthquakes.

The largest historic earthquake was the great California earthquake of April 18, 1906, which had an estimated magnitude of 8.3. Its epicenter was west of South San Francisco, offshore on the San Andreas Fault. Other damaging earthquakes affecting the South San Francisco area occurred in the early and mid-1800s. The more notable of these occurred on the San Andreas Fault in 1838 and 1865.

Another damaging earthquake that affected Bay Area occurred on the Hayward fault in 1868. That earthquake caused considerable damage to buildings on filled ground in Bay Area (Lawson, 1908). A damaging earthquake also occurred on the Calaveras fault in the Dublin area in 1861.

The more recent earthquakes in the region include the February 17, 1989, Loma Prieta earthquake on the San Andreas fault with magnitude of 7.1; the Hollister, Coyote Lake, Morgan Hill , and Alum Rock earthquakes of 1974, 1979, 1984, and 2013 on the Calaveras fault, with magnitudes of 5.2, 5.9, 6.2, and 5.6., respectively; the 1957 Daly City earthquake on the San Andreas fault (magnitude 5.3); and the two Santa Rosa earthquakes of 1969 on the Healdsburg-Rodgers Creek fault (magnitudes 5.6 and 5.7).

6. SUBSURFACE CONDITIONS

Three borings were performed at the location of the subject site by GEC. Previously, Krazan and Associates (KA) performed three additional borings at the site. The approximate locations of the borings is shown on Plate 2, “Site Plan and Boring Location Map” in Appendix-A. In all borings, approximately 1½, to 8 feet of fill material was encountered within the test borings drilled throughout the site by GEC and KA. The fill predominately consisted of gravelly silty sand, clayey silty sand, clayey sand with gravel, or silty sand/sandy silt. The fill was underlain by medium dense to very dense intermixture of clayey silty sand and silty clayey sand to maximum depths of exploration of 61.5 feet in our borings. Below approximately 78 to 100 feet, highly weathered volcanic rock was encountered in KA borings.

Groundwater was encountered in our borings at about 21 to 25 feet by GEC and at about 10½ to 12½ feet below site grades in KA borings. Fluctuations in the groundwater table can be expected with changes in seasonal rainfall, urbanization, and construction activities at or in the vicinity of the site.

This study did not assess contamination of on-site soils and water. A more thorough description and stratification of the soils conditions encountered, along with the results of the laboratory tests, are presented on the respective "Logs of Borings" within Appendix-A.

7. SEISMIC CONSIDERATIONS

Damage to structures related to fault movement may be divided into two categories:

a) Primary deformation such as displacement of a structure located directly on a fault and violent ground shaking, and
b) Secondary failure such as lurch cracking, landsliding, liquefaction, and differential compaction.

Surface faulting or ground rupture tends to occur along lines of previous faulting. Since fault lines are not within the site or project toward the site, the possibility of surface fault rupture is negligible within the subject property.

Ground shaking is a complex concept related to velocity, amplitude, and duration of earthquake vibrations. Damage from ground shaking is caused by the transmission of earthquake vibrations from the ground to the structure. The most destructive effects of an earthquake are usually seen where the ground is unstable and structures are poorly designed and constructed.

Using a 2% probability of exceedance within 50 years, a maximum horizontal ground acceleration of PGA_M = 0.76g was calculated for the site. This calculation considered all active earthquake fault zones within a 100-kilometer radius of the site and a return period of 2,475 years.

The secondary hazards of lateral spreading and lurch cracking are not significant due to the nature of the subsurface materials.

8. CBC EARTHQUAKE DESIGN CRITERIA

The 2013 California Building Code (UBC) Chapter 16, Division IV- Earthquake Design- requires that structures be constructed using certain earthquake design criteria. The criteria are based in part on the seismic zone, soil profile and the proximity of the site to active seismic sources (faults). During an earthquake event, structures located very close to active faults can be subjected to near source energy motions that may be damaging to structures, if the effects of these energy motions are not considered in the structural design. The nearest active fault to the site is the San Andreas fault, which is located approximately 4.7 km northeast of the subject site.

Based on the geotechnical data in this report and the selection of criteria of the 2013 CBC (Chapter 16, Division IV, Earthquake Design), a summary of the earthquake design criteria for use in the design of future structures, additions and improvements is as follows:

TABLE 1- 2013 CBC SESIMIC PARAMETERS

Parameter	Value
Site Class/Soil Profile Type	D
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Mapped MCE Spectral Acceleration (0.2 sec), S_S	1.940
Mapped MCE Spectral Acceleration (1.0 sec), S_1	0.909
MCE Spectral Acceleration (0.2 sec), S_{MS}	1.940
MCE Spectral Acceleration (1.0 sec), S_{M1}	1.363
Design Spectral Acceleration (0.2 sec), S_{DS}	1.294
Design Spectral Acceleration (0.2 sec), S_{D1}	0.909

Although the soils at the site are potentially liquefiable and based on Chapter 20 of ASCE 7, the Site Class for this site should be “F”, however, since the proposed structure has fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class should be determined in accordance with Section 20.3 and the corresponding values of Fa and Fv determined from Tables 11.4-1 and 11.4-2 of ASCE 7. For this site, we determined that the Site Class is “D”.

Based on Section 16 of ASCE 7, for regular structures five stories or less above the base, as defined in Section 11.2 of ASCE 7 and with a period, T, of 0.5 s or less, C_S is permitted to be calculated using the larger of either $S_S = 1.5$ or 80 percent of the value of S_S determined per Section 11.4.1 or 11.4.7 of ASCE 7.

9. LIQUEFACTION POTENTIAL EVALUATION

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils experience a temporary loss of strength due to the build-up of excess pore water pressure during cyclic seismic loadings. In the process, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils most susceptible to liquefaction are loose, clean, saturated, and uniformly graded, fine-grained sands, which occur within about 50 feet or less of the ground surface. Loose, saturated silty and clayey sands may also liquefy during strong ground shaking.

The data used for re-evaluating liquefaction potential of the subsurface soils consisted of data obtained from three (3) exploratory soil borings performed by GEC at locations shown on the Site Plan (Plate 2). We did not performed liquefaction analysis on the boring data obtained by KA since the drilling method was not in accordance with Special Publication 117A.

The main effects of liquefaction at the site include settlement of the ground surface and utilities, lateral deformation, development of excess pore water pressure, buoyancy effects on the below groundwater structures, loss of allowable bearing pressure, and increased lateral pressures on utilities and foundations extending below the groundwater table.

For our liquefaction analysis, we assumed a design groundwater level of 18 feet below grade. Our assumption is based on our findings and the historical groundwater levels presented in the CGS Seismic Hazard Zone report for the Newark Quadrangle (2003).

Our analysis assumed a magnitude (M_w) 7.9 earthquake on the San Andreas Fault and a peak horizontal acceleration of $PGA_M = 0.76g$ according to ASCE 7-10. Furthermore, we assumed that at the time of an earthquake, the groundwater level will be at about 5 feet below the existing ground surface.

For coarse-grained soils such as sand and gravel with various amount of silt and clay, we used a liquefaction evaluation approach developed over the years by Seed and his co-authors.

For fine-grained soils such as silt and clay, there are currently two screening procedures. Both approaches are based on modified Chinese Criteria for liquefaction evaluation of fine-grained soils. The first approach was developed by Bray and Sancio (2006), and another approach was developed by Idriss and Boulanger (2008, 2014). The Bray and Sancio (2006) criteria state that a soil is:

a) Susceptible to liquefaction if $\boldsymbol{w_c/LL > 0.9}$ and $\mathbf{PI < 12}$

b) Moderately susceptible to liquefaction if $\boldsymbol{0.8 < w_c/LL < 0.9}$ and $\mathbf{12 < PI < 18}$

c) Not susceptible to liquefaction if $\boldsymbol{w_c/LL < 0.8}$ and $\mathbf{PI > 18}$

where $\boldsymbol{w_c}$ is water content, LL is Liquid Limit, and PI is Plasticity Index. The criteria presented by Idriss and Boulanger (2008, 2014) state that a soil is

a) sand-like if PI < 7

b) clay-like if PI > 7

where sand-like soils are susceptible to liquefaction, and clay-like soils should be evaluated using Boulanger and Idriss (2004) criteria based on the cyclic triaxial shear testing.

The materials encountered in our borings were sand with various amounts of silt and clay. We treat all the materials as “sand-like”.

Differential compaction occurs when granular subsurface layers above groundwater level settle or compact during earthquakes, with differing amounts of settlement across short horizontal distances. Differential compaction may occur throughout the site in the event of a large earthquake, as even those soils which are not liquefiable are susceptible to densification, and the irregular nature of those loose to medium dense soils will cause varying amounts of settlement to occur. Compaction was calculated along with liquefaction settlement.

Seismically-induced (combination of the liquefaction-induced and differential compaction-induced) settlements at each boring locations analyzed are summarized in Table 2.

Table 2

Estimated Seismically-Induced Settlement

Location	Estimated maximum settlement (inches)
B-1	8
B-2	5
B-3	7

Based on our analysis, we expect seismically-induced settlements of between 5 and 8 inches in the liquefied and unsaturated layers across most of the site. In general, the surface effects of liquefaction in an earthquake are expected to be limited to possible liquefaction-induced differential settlements of up to 5.5 inches across the site due to localized variations in the subsurface profile.

Liquefaction of soils underlying the proposed utilities may also induce temporary buoyant uplift pressures. The magnitude of such pressures is difficult to estimate, because of the variability in materials that may be used and construction techniques. However, given that potentially liquefiable soils are likely only present in continuous layers; it is opinion of GEC that such buoyant uplift pressures would be relatively low.

10. SEISMICALLY-INDUCED LATERAL DEFORMATION

Seismically-induced lateral deformation is another phenomenon which could occur during a seismic event. The continuity/discontinuity of potentially liquefiable soil layers is a key consideration in evaluating the potential for lateral deformation. We evaluated the potential for lateral spreading of the soil using an empirical relationship developed by Youd et al. (2002) and

Zhang et al. (2004). The relationship by Youd et al. (2002) incorporates the thickness of the liquefiable layer, the fines content, mean grain-size of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions, such as a free face, to estimate the horizontal ground movement.

During lateral spreading, surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial soil is transported downslope or in the direction of a free face, such as a channel slope, by earthquake and gravitational forces.

No liquefaction or sign of lateral spreading was noted at the sites during the 1906 San Francisco Earthquake (Youd and Hoose, 1978) or 1989 Loma Prieta Earthquake (USGS, 1990).

Based on the predictive relationship for lateral deformation by Youd et al. (2002) and Zhang et al. (2004), liquefiable soil layer with a blow count of 15 and less may exhibit lateral deformation. The liquefaction analysis results indicate that the potentially non-continuous liquefiable soil layers with blow counts of 15 and less are present between depths of 5 to 10 feet at the site. For significant lateral deformation to occur, a continuous layer of potentially liquefiable soil extending for a considerable distance (on the order of several hundred feet) would be required. Since the liquefaction analysis results indicate the lack of such a layer, it is the opinion of GEC that the potential for lateral deformation at this site would be low.

11. DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

11.1. General

The most prominent geotechnical feature of the site is the presence of undocumented fill and potentially liquefiable soils. The recommendations provided in the following sections will minimize the effects of undocumented fill and potentially liquefiable soils.

From a geotechnical point of view, correction construction of the proposed hotel is feasible provided the recommendations presented in this report are incorporated into the future plans and specifications. This study did not assess contamination of on-site soils and water.

11.2. Recommendations

11.2.1. Site Preparation

Prior to any grading, site preparation of the site should be completed. Site preparation should include the complete removal of all surface and subsurface structures within the footprint of the proposed improvement. Where any of the following are encountered: concrete, septic tanks, pits, gas and oil tanks, storm inlets, foundations, asphalt, machinery, equipment, debris, and trash, these should also be removed with the exception of items specified by the owner for salvage. In addition, all underground structures must be located on the grading plans so that proper removal may be carried out. It is vital that *GEC* intermittently observe the removal of subsurface structures and be notified in ample time to ensure that no subsurface structures are covered. If *GEC* is not contacted to observe the demolition and removal of subsurface structures, then further backhoe investigation will need to be performed prior to the commencement of mass grading.

Excavations made by the removal of any structure should be left open by the demolition contractor for backfill in accordance with the requirements for engineered fill. The removal of underground structures should be done under the observation of the Soil Engineer to assure adequacy of the removal and that subsoils are left in proper condition for placement of engineered fills. Any soil exposed by the demolition operations which are deemed soft or unsuitable by the Soil Engineer, shall be excavated as uncompacted fill or saturated soil and be removed as required by the Soil Engineer during grading. Any resulting excavations should be properly backfilled with engineered fill under the observation of the Soil Engineer. It is important that *GEC* be present during demolition to ensure that all excavations created by grubbing or removal of subsurface structures are left open and located on a grading plan. If any excavations are loosely backfilled without our knowledge and these excavations are not located and backfilled during grading, future settlement of these loosely filled excavations will occur and may cause damage to structures and improvements.

11.2.2. Grading

The grading requirements presented herein are an integral part of the grading specifications presented in Appendix B of this report and should be considered as such.

Grading activities during the rainy season will be hampered by excessive moisture. Grading activities may be performed during the rainy season, however, achieving proper compaction may be difficult due to excessive moisture; and delays may occur. In addition, measures to control potential erosion may need to be provided. Grading performed during the dry months will minimize the occurrence of the above problems.

Undocumented fill materials were encountered across the site. It is recommended that any undocumented fill material encountered within pavement areas, slab-on-grade area and pile cap area, be removed and/or recompacted. The fill materials should be moisture-conditioned to near optimum moisture and recompacted to a minimum of 90 percent of maximum density based on ASTM Test Method D1557.

Following demolition and removal of any loose and/or soft soil, the top 8 inches of exposed native ground for fill areas should be scarified and compacted to a minimum degree of relative compaction of 90% at 1% to 3% above optimum moisture content as determined by ASTM D1557-91 Laboratory Test Procedure. All soils encountered during our investigation are suitable for use as engineered fill when placed and compacted at the recommended moisture content and provided it does not contain any debris.

All engineered fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and compacted to a minimum of 90% relative compaction at 1 to 3 percent above optimum. Relative compaction is based on the maximum dry density as determined by ASTM D1557-91 Laboratory Test Procedure.

Due to planned grading activities it is not anticipated that import soil will be needed for the project.

11.2.3. Surface and Subsurface Drainage

All finish grades should provide a positive gradient to an adequate discharge location in order to provide rapid removal of surface water runoff away from all building foundations. No ponding of water should be allowed on the ground near and adjacent to the foundations. Surface drainage must be provided and maintained at all times.

Lot slopes and drainage must be provided by the project Civil Engineer to remove all storm water from the pad and to minimize storm and/or irrigation water from seeping beneath the structures. Should surface water be allowed to seep under the structures, foundation movement resulting in structural cracking and damage will occur. Finished grades around the perimeter of all residences should be compacted and should be sloped at a minimum 2% gradient away from the exterior foundation. Surface drainage requirements constructed by the builder should be maintained during landscaping. In particular, the creation of planter areas confined on all sides by concrete walkways or decks and the residence foundation is not desirable as any surface water due to rain or irrigation becomes trapped in the planter area with no outlet. If such a landscape feature is necessary, surface area drains in the planter area or a subdrain along the foundation perimeter must be installed.

Continuous roof gutters are recommended. Downspouts from the gutters should be provided with closed pipe conduits to carry storm water away from the structures and graded areas and, thus, reduce the possibility of soil saturation adjacent to the foundations and engineered fills. According to recent state law, roof downspouts drain and flows should be directed to landscape areas where possible. From a geotechnical and maintenance point of view it is undesirable to discharge water into landscape areas near foundations, as these areas generally are not maintained well enough to prevent water ponding. If this must be implemented we recommend that positive drainage away from the foundation is always maintained by the property owners, area drains are located close to the discharge areas to minimize ponding of water and ground cover and vegetation must be maintained to allow easy flow of water to the area drains.

Flower beds or planters are not preferred adjacent to the foundations because of the possibility of irrigation water affecting the foundations. Should planters be constructed, foliage requiring little irrigation should be planted. It is preferred that irrigation adjacent to the building foundations consist of a drip system. Sprinkler systems may be used. However, it is preferred that sprinkler heads do not water closer than 3 feet from the building foundations. If sprinklers are used within 3 feet, then excessive watering should not be allowed and good surface drainage in the planter area must be provided. In any case, it is recommended that area surface drains be incorporated into the landscaping to discharge any excessive irrigation or rainwater that may accumulate in the planter area. These surface drains must be constructed such that the surface of the drain is lower than the surrounding grade so that easy flow of surface water runoff is allowed into the drip inlets.

11.2.4. Liquefaction Mitigation

Seismically-induced settlements of up to 8 inches were estimated at the site. The designers should either design for such settlements, or where estimated seismically-induced settlements cannot be tolerated, they should be mitigated through a program of ground improvement. There are several techniques available for soil improvement which may be applicable to this site: vibro-replacement stone columns, grouting techniques. Alternatively, the liquefaction-induced settlement can be minimized by supporting the structure on driven piles. A low-vibration piling system (such as Screw-in Piling or Press-in Piling) may be used where vibration due to pile driving activities cannot be tolerated. In these low-vibration systems, a pile is screwed or pushed into the strata, with the resulting skin friction and end bearing capacities similar to driven piles. We understand that you may use vibration-free pile foundation for this project to avoid any damage to the neighboring structures and reduce construction-induced noise impact. However, we included this section for sake of completeness.

This section provides several feasible options for liquefaction mitigation measures. Ground improvement should be performed in areas where the total calculated seismically-induced settlement exceeds the structurally acceptable level, and be designed to reduce total liquefaction-induced settlement to a tolerable level. The soil zone to be improved includes those soils which are at depth of 5 to 45 feet. The total thickness of the zone to be improved depends both on the actual thickness of the soil layer and the desired reduction in predicted settlement. GEC anticipates that after improvement of soils to depth of 45 feet, the seismically-induced settlement will reduce to less than 1-inch or less.

The vibro-replacement stone column technique of ground treatment has proven successful in reducing the liquefaction potential of sands and low plasticity silt. Stone columns are used for loose silty sands having greater than about 15 percent fines. Cohesive, mixed and layered soils generally do not densify easily when subjected to vibration alone, therefore, the vibro-replacement stone column technique was developed specifically for these soils, effectively extending the range of soil types that can be improved with the deep vibratory process.

Grouting techniques (compaction, permeation, deep mixing, chemical, and jet grouting) of soil improvement have also proven successful in reducing the liquefaction potential of sandy material. The grouting techniques become less efficient with increased fines content, such as silt and clay. Of these grouting techniques, compaction grouting appears to be the most economic method for the site. Essentially, in compaction grouting, the injection of a stiff mortar-like cement grout into a compatible soil mass will achieve controlled densification of that mass by physically moving the soil particles, radially from a growing bulb of grout into a closer spacing. Other grouting techniques, such as deep mixing, involve the use of large augers both to introduce cement grout and to mix it with the soil, producing a treated soil cement column.

The soil improvement design will depend on the costs of performing the work as well as the technical specifics of the work, and is beyond the scope of this study.

11.2.5. Foundations

11.2.5.1. Driven Piles

We recommend that the proposed hotel to be supported on 12-inch steel pipe screw-in pile foundation system.

Pile foundations should (a) resist (with appropriate factor of safety) the vertical (uplift and downward) loads due to dead loads, operating live loads, and seismic loads; (b) resist lateral loads induced by the structure with acceptable lateral displacements; (c) resist the lateral loads on the piles due to movement of the soil during the earthquake; and (d) limit vertical differential settlements in the structure to acceptable level.

It is our opinion that the 12-inch steel pipe piles screwed to at least 60 feet below the existing ground surface or to practical refusal will achieve a static allowable compression capacity of 70 tons. The allowable compression capacity includes a factor of safety of two and an estimated downdrag load of 20 tons induced by the static and seismically-induced settlements. Therefore, no additional reductions are necessary to account for downdrag.

The ultimate uplift capacity for a single pile driven to the above-recommended depth is estimated to be 25 tons. A factor of safety of two is recommended to calculate allowable uplift capacities. Additional uplift resistance may be incorporated into design by adding the buoyant weight of the piles to the recommended uplift capacity of the piles.

Resistance to horizontal forces induced by seismic shaking or other factors may be provided by passive pressure acting against the buried pile cap and piles and by sliding friction acting on the bottom of the foundation. For this project, the available passive pressure may be calculated using the equivalent fluid unit weights given in Table 5 and ignoring the upper 2 feet for the materials above groundwater level. For piles, the designer may use Plate 7. These values include a factor of safety of 1.5. The lateral load capacities presented on Plate 7 were developed assuming a free head condition, no group influences, and a maximum vertical load of 70 tons, hollow steel Grade 60 pipe pile.

Friction along the sides of the pile caps, grade beams, and slab-on-grades may be used in combination with the allowable passive resistance. The effective at-rest pressures of 65 pcf (above groundwater level) normal to the sides of the structural elements should be used in estimating frictional resistance along the sides.

TABLE 5
LATERAL LOAD RESISTANCE PARAMETERS

Parameter	
Passive Equivalent Fluid Unit Weight (pcf)	300
Ultimate Frictional Coefficient*	0.35

*Coefficient of friction between soil and concrete. The coefficient of friction will reduce to 0.3 if wood lagging left in place.

The resistance from the upper 12 inches of soil beneath the ground surface should be neglected in lateral capacity calculations; however, the pressure distribution should be calculated from the surface.

Both the above-recommended friction and passive pressure values assume similar deflections in order to mobilize the lateral resistance; therefore, lateral friction and passive pressure may be used in combination without reduction.

The above pile capacities are based on the strength of the soil and the structural capacities of the pile depends on the strength of the pile materials which should be checked by the structural engineer. Piles should be spaced at least three times the pile width, center to center. The allowable pile capacity may be reduced by group action when space between two adjacent piles is closer than three times the width of the new piles, and where this occurs additional geotechnical analyses will be necessary.

We did not evaluate corrosion potential of the soils. Based on our experience in the area, it is our opinion that the soils are “moderately corrosive”. The moderately ccorrosive soils may adversely affect the steel piles. The adverse effects may be mitigated for steel piles by considering sacrificial thickness in the calculations. Alternative, the piles may be coated with bituminute materials which adversely reduce downward and uplift capacities. Pile caps will be in the fill or above existing ground surface. Use of sulfate-resistant high density/low porosity cement, and providing a good coverage of mortar over the concrete pile reinforcements should be considered for the concrete pile caps.

It is recommended that an indicator pile program be undertaken to ascertain the driving resistance and verify the pile capacities at the structure location across the site and to obtain field data for the selection of production pile lengths. Monitoring of pile driving using a Pile Driving Analyzer (PDA) during the indicator program is recommended to evaluate refusal criteria, to ascertain the stresses in the pile during driving, to estimate damage to the new and existing piles during driving, and to develop additional data as to the ultimate pile capacity. We recommend that an indicator pile program of at least 6 piles be performed for the proposed hotel. The piles should be driven using the same equipment planned to install production piles. The contractor is responsible to select the equipment based on the geotechnical borings presented in this report or obtaining additional borings. The indicator piles may be used as production pile after review of PDA results.

In addition, undocumented fill exists at the site. Nature of the materials within the fill is very heterogeneous and may contains rubble and construction debris which causes refusal during pile driving. Therefore, it is recommended that predrilling within the fill material be performed as a minimum. The predrilled holes should have a diameter of 12 inches to prevent substantial reduction in frictional capabilities of the pile. The depth of the predrilled holes should be a maximum of 20 feet below the existing surface. Predrilling below groundwater level may require casing.

Settlements of the piles screwed into the lower stiff clay and/or dense to very dense sands are estimated to be less than ½ inch for the design loads discussed above. Differential settlements between adjacent piles are estimated to be less than ¼ of an inch. These settlement values do not include axial compression of the piles under design loads.

Differential settlements between the pile-supported structure and surrounding areas may occur during and following a major earthquake as a result of seismically-induced settlements of the loose to medium dense sandy fill soils as discussed above. Therefore flexible connections and pipe are recommended to reduce damage during a major earthquake event.

11.2.5.2. Structural Floor Slabs

The proposed pile-supported structures are underlain by weak fill, and liquefiable soils are not suitable for support of slabs-on-grade. Therefore, we recommend that structural floor slabs be used for the pile-supported structures.

Slab subgrades should be rolled smooth prior to construction to provide a uniform dense, non-yielding surface. A capillary break consisting of 4 inches of clean, open-graded, ¾-inch gravel should be placed beneath slab-on-grade floors and structural floor slabs. This material should be compacted with a vibratory roller and should conform to the gradation criteria presented in Table 3.

TABLE 3
RECOMMENDED CAPILLARY BREAK MATERIAL

Sieve Size (U.S. Series)	Percentage Passing
1 inch	100
¾ inch	90 - 100
No. 8	0 - 10
No. 16	0 - 5
No. 200	0 - 3

For those situations where moisture condensation is a likely problem, a moisture barrier should be provided below slab-on-grade floors and suspended structural floor slabs. This would include two inches of clean-washed sand overlying a Class A visqueen membrane such as Stego Wrap 15mil or approved equal.

11.2.6. Exterior Concrete Flatwork Slab-on-Grade Construction

Small equipment pad, sidewalks and other minor improvements may be supported on slab-on-grade. At least 24 inches of the subgrade should be removed and replaced with non-expansive engineered fill compacted to minimum 95 percent relative compaction. It is expected that the exterior concrete floor slabs-on-grade may experience some cracking of the soil on the site. To reduce the potential cracking of concrete, the following are recommended:

- The slab subgrade saturation is anticipated prior to pouring the slab. If the finished pad is determined to be expansive by our representative, additional recommendations will be required.

- Slabs should be underlain by a minimum of 4 inches of angular gravel or clean crushed rock material placed between the finished subgrade and the slabs to serve as a capillary break between the subsoil and the slab. See the "Guide Specifications For Rock Under Floor Slabs", Appendix B.

- We note that some 4” thick slab-on-grade foundation systems with conventional footings have experienced excessive cracking. In order to provide better slab performance with respect to cracking, a slab thicker than 4 inches reinforced with reinforcement bars must be used. We recommend that the slabs be a minimum 5 inches thick and be reinforced with a minimum of No. 3 bars spaced 18 inches center to center, each way or as determined by the project structural engineer for the anticipated floor loads. The reinforcement shall be placed in the center of the slab unless otherwise designated by the structural engineer. Alternatively, the slab may be reinforced with welded wire fabric sheets. Wire mesh must not be used for reinforcement. The project structural engineer will design the floor slab thickness and actual reinforcement.

- Where floor coverings are anticipated, a 10-mil or thicker Visqueen-type membrane should be placed between the rock cushion and the slab to provide an effective vapor retarder and to minimize moisture condensation under the floor covering. It is suggested that a two inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete and to prevent puncture of the membrane.

- Slabs at door openings should be constructed with a curl or a thickened edge extending a minimum of 12 inches into native ground or compacted fill.

- A minimum of two inches of moistened sand should be placed over the vapor barrier to facilitate curing of the concrete and to act as a cushion to protect the membrane. The perimeter of the slab should be thickened to bear on the prepared building pad and to confine the sand. During winter construction, sand may become saturated due to rainy weather prior to pouring. Saturated sand is not desirable because there exists a high probability of creating sand pockets within the slab section during the concrete pour. As an alternate, a sand-fine gravel mixture that is stable under saturated conditions may be used. However, the material must be approved by the Special Inspector prior to use.

- Since the foundation subgrade will consist of clayey material, saturation of slab subgrade prior to pouring is needed. The upper 12 inches of subgrade should be compacted to 90 percent with moisture content of 2 to 4 percent above the optimum moisture content as determined by ASTM 1557. The slab subgrade should be wetted to seal the cracks. In this case the soil engineer should observe and verify the subgrade soil wetting before the slabs are poured.

- It is expected that the concrete slabs-on-grade including public sidewalks, driveways and other landscape flatwork may experience some cracking due to the expansive nature of the soil on the site. To reduce the potential cracking of concrete, the following are recommended:

  a. To decrease the amount of potential swelling, the driveway subgrade soil in the upper 12 to 18 inches of the subgrade shall be saturated until a moisture equilibrium is achieved (minimum 5% above optimum moisture) before the slab is poured. The Soil Engineer should observe and verify the subgrade soil saturation before the slabs are poured. Typically, 12 inches penetration with a thin metal probe may indicate sufficient saturation. The subgrade for other flatwork slabs should be thoroughly wetted prior to the pouring of concrete.

  b. Driveway slabs should be a minimum 4 inches in thickness and be underlain by a minimum of 4 inches of crushed gravel over subgrade. The perimeter edge of the driveway slab may be constructed with an 8 inch thickened perimeter edge to contain the gravel and minimize the potential future migration of surface water into the driveway subgrade from the adjoining landscape area. See the "Guide Specifications for Rock under Floor Slabs", Appendix B. Alternatively a thicker slab without gravel may be used. Concrete flatwork for walkways can be cast directly on prepared subgrade and the typical layer of crushed gravel between the flatwork and subgrade can be omitted.

c. The flatwork and driveway slabs should be reinforced at a minimum with welded wire fabric sheets and not wire mesh. Reinforcing bars may also be used, if desired. Reinforcement is to be placed in the center of the slab by utilizing chairs or other equivalent support systems unless otherwise designated by the design engineer. Slabs should be properly reinforced to meet structural design criteria. The actual reinforcement to use is to be determined by others.

d. All exterior flatwork slabs such as steps, patios, or sidewalks should be poured structurally independent of the foundations. A 30-pound felt strip, expansive joint material, or other positive separator should be provided around the edge of all floating slabs to prevent bond to the structure foundation.

11.2.7. <u>Pavement Recommendation</u>

We understand that the proposed project includes driveway and parking lots. Roadway is expected to have flexible pavement.

At the time of preparation of this report, traffic indices for the proposed driveway pavement was not available to us. Therefore, we has developed recommendations for flexible pavement for several traffic indices (TI) as tabulated below.

The pavement components should be designed and constructed using the latest Caltrans specifications and procedures. KA performed two R-value tests with values ranging from 32 to 37. Due to variability of the materials within the on-site materials, the pavement sections in Table 4 assume a subgrade R-value of 25. Compaction of the pavement components should be to at least 95 percent relative compaction, in accordance with the ASTM D 1557 procedure.

TABLE 4

SUGGESTED FLEXIBLE PAVEMENT SECTIONS

TI	AC Thickness (in)	AB[1] Thickness (in)
4.0	3	4
4.5	3	5
5.0	3	6.5
5.5	4	6
6	4	7.5
7	4	11

Notes: Caltrans Highway Design Manual, Chapter 630 (2013):
GE = Gravel Equivalent for Pavement Section;
AB = Aggregate Base (Min.R-Value = 78);
AC = Hot Mix Asphalt Concrete;
Factor of safety included. Section thickness rounded to the nearest inch.
Aggregate subbase or recycled baserock meeting CALTRANS subbase requirements could be substituted for CALTRANS Class 2 AB using a GE factor of 1.1 times that of AB for a slightly thicker section.

11.2.8. Utility Trenches

Applicable safety standards require that trenches in excess of 5 feet must be properly shored or that the walls of the trench slope back to provide safety for installation of lines. If trench wall sloping is performed, the inclination should vary with the soil type. The underground contractor should request an opinion from the Soil Engineer as to the type of soil and the resulting inclination.

With respect to state-of-the-art construction or local requirements, utility lines are generally bedded with granular materials. These materials can convey surface or subsurface water beneath the structures. It is, therefore, recommended that all utility trenches which possess the potential to transport water be sealed with a compacted impervious cohesive soil material or lean concrete where the trench enters/exits the building perimeter. This impervious seal should extend a minimum of 2 feet away from the building perimeter.

Utility trenches extending underneath all traffic areas must be backfilled with native or approved import material and compacted to relative compaction of 90% to within 6 inches of the subgrade. The upper 6 inches should be compacted to 95% relative compaction in accordance with Laboratory Test Procedure ASTM D1557-91. Backfilling and compaction of these trenches must meet the requirements set forth by the City of South San Francisco, Department of Public Works. Utility trenches within landscape areas may be compacted to a relative compaction of 85%.

11.2.9. Coefficient of Permeability

We evaluated the coefficient of permeability of the soils based on the results of laboratory grain size distribution (sieve analyses) tests.

The existing fill is very heterogeneous and contains up to 40 percent fines (silt plus clay) by weight. The anticipated range of coefficient of permeability for these soil types, classified as clayey silty sand with gravel, is 10^{-3} to 10^{-6} centimeters per second (cm/sec).

11.2.10. Construction Considerations

11.2.10.1. Effects on Adjacent Facilities

During excavations adjacent to existing structures and electric towers, care should be taken to adequately support facilities that might be affected by the proposed construction procedures. During excavation below the foundation level of adjacent structures underpinning may be required if excavations extend below an imaginary plane sloping at 1H:1V downward and outward from the edge of existing foundations.

11.2.10.2. Vibrations During Pile Driving

Since a screw-in piling system is used, vibration is not of any concern. If different piling method is utilized. Effect of pile-driving-induced vibration on the adjacent structures should be considered. In this case, during pile driving, the effects of vibrations should be closely monitored in structures adjacent to the proposed construction. The potential for damage or distress depends on the size of the hammer, the energy delivered, the distance to the adjacent structures, and the type of foundations supporting those structures.

Evaluating the structural damage potential typically consists of the following steps:

- Development of damage criteria and initial assessment of damage potential;

- Design of appropriate instrumentation to measure ground particle velocities and survey movements;
- Measurement of response, such as maximum velocity and predominant frequency of vibration; and
- Comparison of the measured quantities with those permitted according to the damage criteria.

11.2.10.3. Sequence of Filling and Pile Driving

The conventional sequence for pile construction is for the fill to be placed and compacted to the design subgrade level and the piles driven from the subgrade level. However, where structures have depressed slabs, mobility and access to the subgrade may be restricted. For this condition the piles are driven using a follower to the design cut-off elevation from a temporary surface above the design subgrade. After all the piles are driven the site is excavated and the pile butts exposed and construction continues in the usual fashion. Typically the selection of the most appropriate method and sequence of pile construction is the responsibility of the contractor.

11.2.11. Project Review and Construction Monitoring

All grading and foundation plans for the development must be reviewed by the Soil Engineer prior to contract bidding or submitted to governmental agencies so that plans are reconciled with soil conditions and sufficient time is allowed for suitable mitigative measures to be incorporated into the final grading specifications.

GEC should be notified at least two working days prior to site clearing, grading, and/or foundation operations on the property. This will give the Soil Engineer ample time to discuss the problems that may be encountered in the field and coordinate the work with the contractor. Field observation and testing during foundation operations must be provided by representatives of *GEC*, to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to the grading and/or foundation operations performed without the full knowledge and under the direct observation of the Soil Engineer will render the recommendations of this report invalid. The degree of observation and frequency of testing services would depend on the construction methods and schedule, and the item of work. Please refer to "Guidelines For Required Services" for an outline of our involvement during project development.

Should another geotechnical consultant be engaged to perform project review and/or construction monitoring, then *GEC*, must receive a letter of indemnification releasing us of any responsibility on the project.

Should you have any questions relating to the contents of this report or should additional information be required, please contact our office at your convenience.

Sincerely,

GEC

REGISTERED PROFESSIONAL ENGINEER
KAMRAN GHIASSI
No. C66379
No. GE2792
Exp. 6/30/2016
CIVIL & GEOTECHNICAL
STATE OF CALIFORNIA

K. Ghiassi

Kamran Ghiassi, Ph.D., G.E.
Registered Geotechnical Engineer

12. GUIDELINES FOR REQUIRED SERVICES

The following list of services is the services required and must be provided by *GEC* during the project development. These services are presented in check list format as a convenience to those entrusted with their implementation.

The items listed are included in the body of the report in detail. This list is intended only as an outline of the required services and does not replace specific recommendations and, therefore, must be used with reference to the total report. This does not imply full-time observation. The degree of observation and frequency of testing services would depend on the construction methods and schedule, and the item of work.

The importance of careful adherence to the report recommendations cannot be overemphasized. It should be noted, however, that this report is issued with the understanding that each step of the project development will be performed under the direct observation of *GEC.*

The use of this report by others presumes that they have verified all information and assume full responsibility for the total project.

	Item Description	Required	Not Required
1.	Provide foundation design parameters	X	
2.	Review grading plans and specifications	X	
3.	Review foundation plans and specifications	X	
4.	Observe and provide recommendations regarding demolition	X	
5.	Observe and provide recommendations regarding site stripping	X	
6.	Observe and provide recommendations on moisture conditioning, removal, and/or precompaction of unsuitable existing soils	X	
7.	Observe and provide recommendations on the installation of subdrain facilities	X	
8.	Observe and provide testing services on fill areas and/or imported fill materials	X	
9.	Review as-graded plans and provide developmental foundation recommendations, if necessary	X	
10.	Observe and provide compaction tests on sanitary sewers, storm drain, water lines and PG&E trenches	X	
11.	Observe foundation excavations and provide supplemental recommendations, if necessary prior to placing concrete	X	
12.	Observe and provide moisture conditioning recommendations for foundation areas prior to placing concrete	X	
13.	Provide design parameters for retaining walls	X	
14.	Provide observations and recommendations for keyway excavations and cutslopes during grading		X
15.	Excavate and recompact all geologic trenches and/or test pits		X
16.	Observe installation of subdrains behind retaining walls		X

13. LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his representative to notify *GEC* in writing, a minimum of two working days before any clearing, grading, or foundation excavations can commence at the site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings and from a reconnaissance of the site. Should any variations or undesirable conditions be encountered during the development of the site, *GEC*, will provide supplemental recommendations as dictated by the field conditions.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

4. At the present date, the findings of this report are valid for the property investigated. With the passage of time, significant changes in the conditions of a property can occur due to natural processes or works of man on this or adjacent properties. In development, legislation or the new knowledge may result in changes in applicable standards. Changes outside of our control may render this report invalid, wholly or partially. Therefore, this report should not be considered valid after a period of two (2) years without our review, nor should it be used, or is it applicable, for any properties other than those investigated.

5. Not withstanding, all the foregoing applicable codes must be adhered to at all times.

APPENDIX A

Field Investigation

Site Vicinity Map

Site Plan and Boring Location Map

Logs of Test Borings

Lateral Pile Analysis Results

FIELD INVESTIGATION

The field investigation was performed on February 5th, 2016 and included a reconnaissance of the site and the drilling of three (3) test boring at the approximate locations shown on Plate 2, "Site Plan and Boring Location Map."

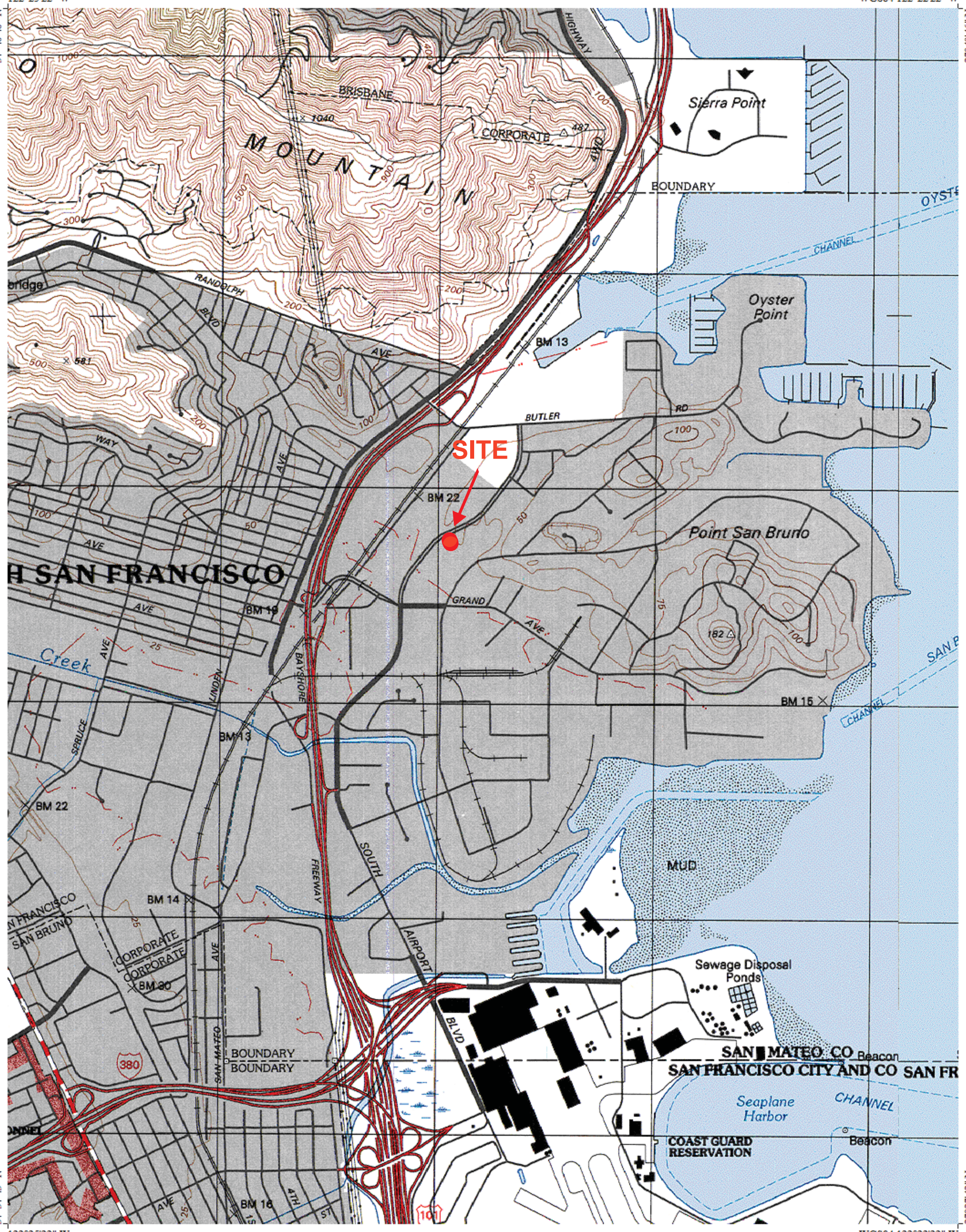
The borings were drilled to maximum depths of 61.5 feet below the existing ground surface. The drilling was performed with a truck-mounted drill rig equipped with a 4.5-inch. The borehole was drilled using rotary wash drilling method. Visual classifications were made from the auger cuttings and the samples in the field. As the drilling, disturbed core samples were obtained. Classifications made in the field were verified in the office after further examination.

The stratification of the soils and descriptions are shown on the "Logs of Test Borings" contained within this appendix.

TOPO! map printed on 02/15/16 from "SFBAY.tpo" and "Untitled.tpg"

122°25'22" W | WGS84 122°22'22" W

37°40'46" N



37°37'43" N

122°25'22" W | WGS84 122°22'22" W

TN / MN 15°

0 | 1/2 | 1 MILE

0 | 1000 FEET | 0 | 500 m | 1000 m

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N

SITE VICINITY MAP

550 GATEWAY BOULEVARD
SOUTH SAN FRANCISCO, CALIFORNIA

GeoEngineering Consultants

JOB NO. P16-0122	DATE: 11/15	PLATE 1

B-1
Approximate Boring Locations by Krazan

B-1
Approximate Proposed Boring Locations

The image is courtesy of Google Earth Pro. The scale is shown on the upper left corner of the image.

SITE PLAN AND BORING LOCATION MAP

550 GATEWAY BOULEVARD
SOUTH SAN FRANCISCO, CALIFORNIA

DATE: 11/15

PLATE 2

PROJECT:	550 Gateway Blvd. South San Francisco, CA	Log of B-1

BORING LOCATION:	See site plan	ELEVATION AND DATUM: ---	
DRILLING CONTRACTOR:	Geo_Ex Drilling	DATE STARTED: 2/5/2016	DATE FINISHED: 2/5/2016
DRILLING METHOD:	Rotary Wash	TOTAL DEPTH (ft.): 61.5	MEASURING POINT: Ground Surface
DRILLING EQUIPMENT:	CME 55	DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED: 21	
SAMPLING METHOD:	Modified California & SPT samplers	DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs	DROP: 30 inches	LOGGED BY: YB

DEPTH (feet)	SAMPLES Sample No.	Blows/ Foot	MATERIAL DESCRIPTION	LABORATORY TESTS Moisture Content (%)	Dry Density (pcf)	Other
0			Clayey Silty SAND with Gravel (SW), reddish brown to brown, moist, loose to medium dense, fine subangular gravel (FILL)			
	1	12,17,19		9	115	-#200=21% LL=23% PI=7%
5	2	6,6,5				
			Clayey Silty SAND (SM) yellowish brown to reddish brown with red mottling, moist, medium dense to dense, fine to medium grained sand.			
10	3	13,26,50		7	117	-#200=27%
15	4	20,25,30				
20	5	6,9,10	Silty Clay SAND (SC) yeddish yellow, wet, medium dense, fine to medium grained sand.	12		
25	6	8,9,11		18	121	
30	7	5,6,9		16		
35						

PROJECT:	550 Gateway Blvd. South San Francisco, CA	Log of B-1	
BORING LOCATION:	See site plan	ELEVATION AND DATUM: ---	
DRILLING CONTRACTOR:	Geo_Ex Drilling	DATE STARTED: 2/5/2016	DATE FINISHED: 2/5/2016
DRILLING METHOD:	Rotary Wash	TOTAL DEPTH (ft.): 61.5	MEASURING POINT: Ground Surface
DRILLING EQUIPMENT:	CME 55	DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED: 21	
SAMPLING METHOD:	Modified California & SPT samplers	DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs DROP: 30 inches	LOGGED BY: YB	

DEPTH (feet)	Sample No.	Sample	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
	8		10,11,9				
				Clayey Silty SAND (SM) reddish brown, wet, dense to very dense, fine to medium grained sand.			
40	9		10,18,25		17		
45	10		14,15,15				
50	11		14,19,18		18		
55							
60	12		15,24,34		17		
				Boring was terminated at about 61.5 feet. Boring backfilled with cement grout.			

PROJECT:	550 Gateway Blvd. South San Francisco, CA	Log of B-2	
BORING LOCATION:	See site plan	ELEVATION AND DATUM: ---	
DRILLING CONTRACTOR:	Geo_Ex Drilling	DATE STARTED: 2/5/2016	DATE FINISHED: 2/5/2016
DRILLING METHOD:	Rotary Wash	TOTAL DEPTH (ft.): 61.5	MEASURING POINT: Ground Surface
DRILLING EQUIPMENT:	CME 55	DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED: 22	
SAMPLING METHOD:	Modified California & SPT samplers	DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs DROP: 30 inches	LOGGED BY: YB	

DEPTH (feet)	Sample No.	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
0	1	12,17,19	Clayey Silty SAND with Gravel (SW), reddish brown to brown, moist, loose to medium dense, fine subangular gravel (FILL)			
5	2	6,6,5	Clayey Silty SAND (SM) yellowish brown to reddish brown with red mottling, moist, medium dense to dense, fine to medium grained sand.	11	119	-#200=38% LL=23% PI=7%
10	3	13,26,50				
15	4	20,25,30				
20	5	6,9,10		16		LL=22% PI=7%
			Silty Clay SAND (SC) yeddish yellow, wet, medium dense, fine to medium grained sand.			
25	6	8,9,11				
30	7	5,6,9		17		LL=26% PI=11%
35						

PROJECT:	550 Gateway Blvd. South San Francisco, CA	Log of B-2	
BORING LOCATION:	See site plan	ELEVATION AND DATUM: ---	
DRILLING CONTRACTOR:	Geo_Ex Drilling	DATE STARTED: 2/5/2016	DATE FINISHED: 2/5/2016
DRILLING METHOD:	Rotary Wash	TOTAL DEPTH (ft.): 61.5	MEASURING POINT: Ground Surface
DRILLING EQUIPMENT:	CME 55	DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED: 22	
SAMPLING METHOD:	Modified California & SPT samplers	DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs DROP: 30 inches	LOGGED BY: YB	

DEPTH (feet)	Sample No.	Sample	Blows/Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
	8		10,11,9				
				Clayey Silty SAND (SM) reddish brown, wet, dense to very dense, fine to medium grained sand.			
40					18		
	9		10,18,25				
45	10		14,15,15				
50					15		
	11		14,19,18				
55							
60	12		15,24,34				
				Boring was terminated at about 61.5 feet. Boring backfilled with cement grout.			

Log of B-3

PROJECT:	550 Gateway Blvd. South San Francisco, CA
BORING LOCATION:	See site plan
ELEVATION AND DATUM:	---
DRILLING CONTRACTOR:	Geo_Ex Drilling
DATE STARTED:	2/5/2016
DATE FINISHED:	2/5/2016
DRILLING METHOD:	Rotary Wash
TOTAL DEPTH (ft.):	61.5
MEASURING POINT:	Ground Surface
DRILLING EQUIPMENT:	CME 55
DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED:	25
SAMPLING METHOD:	Modified California & SPT samplers
DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs
DROP:	30 inches
LOGGED BY:	YB

DEPTH (feet)	Sample No.	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
0			Clayey Silty SAND with Gravel (SW), reddish brown to brown, moist, loose to medium dense, fine subangular gravel (FILL)			
	1	18,22,26		9	117	
			Clayey Silty SAND (SM) yellowish brown to reddish brown with red mottling, moist, medium dense to dense, fine to medium grained sand.			
5	2	8,8,10		11	119	-#200=28% LL=26% PI=11%
10	3	10,9,14		14		
15	4	9,16,18		15	123	
20	5	11,15,20				
			Silty Clay SAND (SC) yeddish yellow, wet, medium dense, fine to medium grained sand.			
25	6	6,10,18		16		LL=35% PI=18%
30	7	8,16,25		18		
			Clayey Silty SAND (SM) reddish brown, wet, dense to very dense, fine to medium grained sand.			
35						

Log of B-3

PROJECT:	550 Gateway Blvd. South San Francisco, CA
BORING LOCATION:	See site plan
ELEVATION AND DATUM:	---
DRILLING CONTRACTOR:	Geo_Ex Drilling
DATE STARTED:	2/5/2016
DATE FINISHED:	2/5/2016
DRILLING METHOD:	Rotary Wash
TOTAL DEPTH (ft.):	61.5
MEASURING POINT:	Ground Surface
DRILLING EQUIPMENT:	CME 55
DEPTH TO WHERE FREE WATER FIRST ENCOUNTERED:	25
SAMPLING METHOD:	Modified California & SPT samplers
DEPTH TO WATER AT COMPLETION:	
HAMMER WEIGHT:	140 lbs
DROP:	30 inches
LOGGED BY:	YB

DEPTH (feet)	Sample No.	Blows/ Foot	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	Other
	8	7,10,11		17		LL=20% PI=7%
40				17		
	9	7,10,15				
45	10	11,23,28				
50				17		
	11	13,22,40				
55						
60	12	43,50/2"				
			Boring was terminated at about 61.5 feet. Boring backfilled with cement grout.			

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			SOIL SYMBOL	SOIL DESCRIPTION
COARSE GRAINED SOILS More than half material is larger than # 200 sieve	GRAVELS (More than 50 % material larger than # 4 sieve)	CLEAN GRAVEL Less than 5%fines	GW	Well Graded Gravels, Gravel-Sand Mixtures, little or Fines
			GP	Poorly Graded Gravels or Gravel-Sand Mixtures, little or No Fines
		GRAVEL With Fines (More than 12% fines)	GM	Silty Gravels, Gravel-Sand-Silt Mixtures, Non-Plastic Fines.
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures, Plastic Fines.
	SANDS (More than 50 % material smaller than # 4 sieve)	CLEAN SAND (Less than 5% fines)	SW	Well Graded Sands, Gravelly Sands, Little or No Fines.
			SP	Poorly Graded Sands or Gravelly Sands, Little or No Fines.
		SAND With Fines (More than 12% fines)	SM	Salty Sands, Sand-Silt Mixtures, Non-Plastic Fines.
			SC	Clayey Sands, Sand-Clay Mixtures, Plastic Fines.
FINE GRAINED SOILS More than half material is smaller than the #200 sieve	SILTS & CLAYS Liquid Limit is less than 50%		ML	Inorganic Silts, Sandy or Clayey Silts, Low to no Plasticity.
			CL	Inorganic Clay, Sandy or Silty Clay, Low to Medium Plasticity.
			OL	Organic Silt or Organic Silty Clay, Low to Medium Plasticity.
	SILTS & CLAYS Liquid limit is greater than 50%		MH	Inorganic Silts, Diatomaceous or Micaceous, Fine Sandy or Silty Soils.
			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.

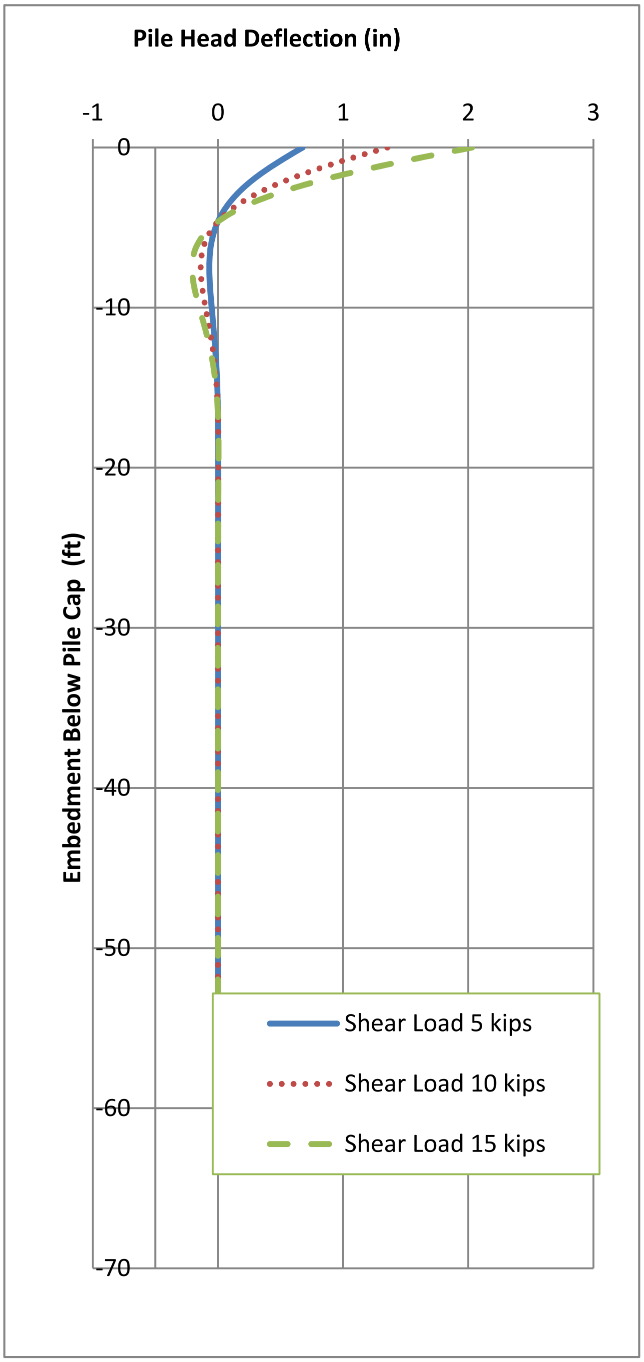
PARTICLE SIZE LIMITS

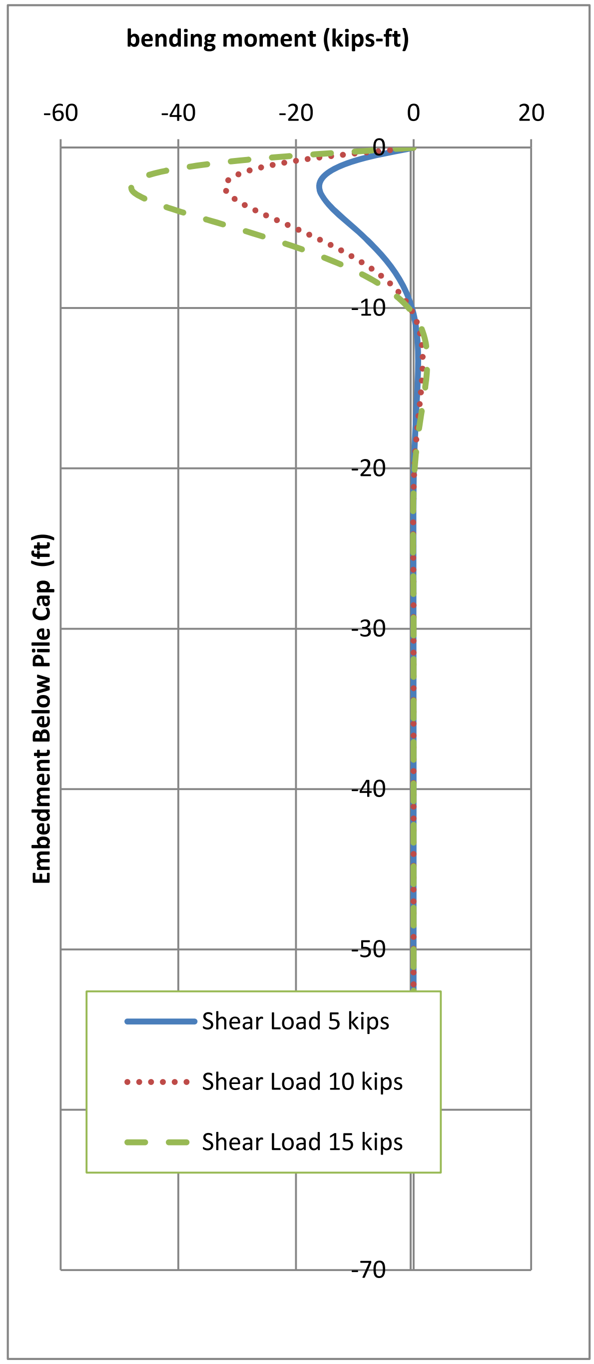
(Sieve Openings in mm.)	.074	.425	2.00	4.17	19.0	75.0	300.0	
SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS	
	FINE	MEDIUM	COARSE	FINE	COARSE			
(U.S. Standard sieve Sizes)	# 200	# 40	# 10	# 4	.75 in	3 in	12 in	

RELATIVE DENSITY		CONSISTENCY	
SANDS, GRAVELS AND NON- PLASTIC SILTS	**BLOWS / FOOT***	CLAYS AND PLASTIC SILTS	STRENGTH†
VERY LOOSE	0 - 4	VERY SOFT	0 - 1/ 4
LOOSE	4 - 10	SOFT	1/4 - ½
MEDIUM DENSE	10 - 30	FIRM	1/2 - 1
DENSE	30 - 50	STIFF	1 - 2
VERY DENSE	OVER 50	VERY STIFF	2 - 4
		HARD	OVER 4

* Numbers of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1 – 3 / 8 inch I. D.) split spoon (ASTM D -1586).
† Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the Standard Penetration test (ASTM D – 1586), pocket penetrometer, torvane or visual observation.

GeoEngineering GEC Consultants	**GeoEngineering Consultants**	**KEY TO EXPLORATORY BORING LOGS**



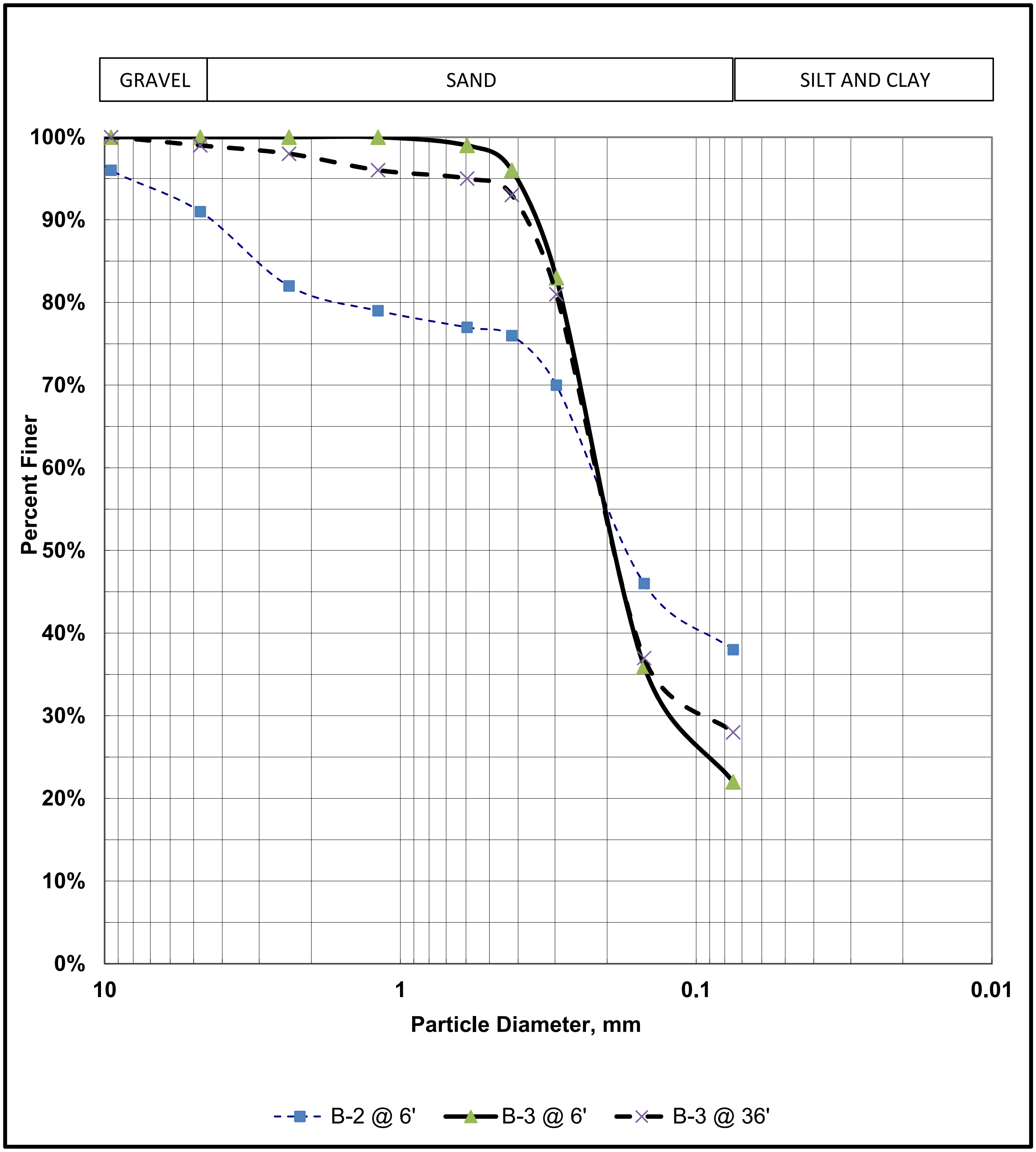


LATERAL LOAD ANALYSIS
Free Head Condition - 12-Inch Pipe Pile

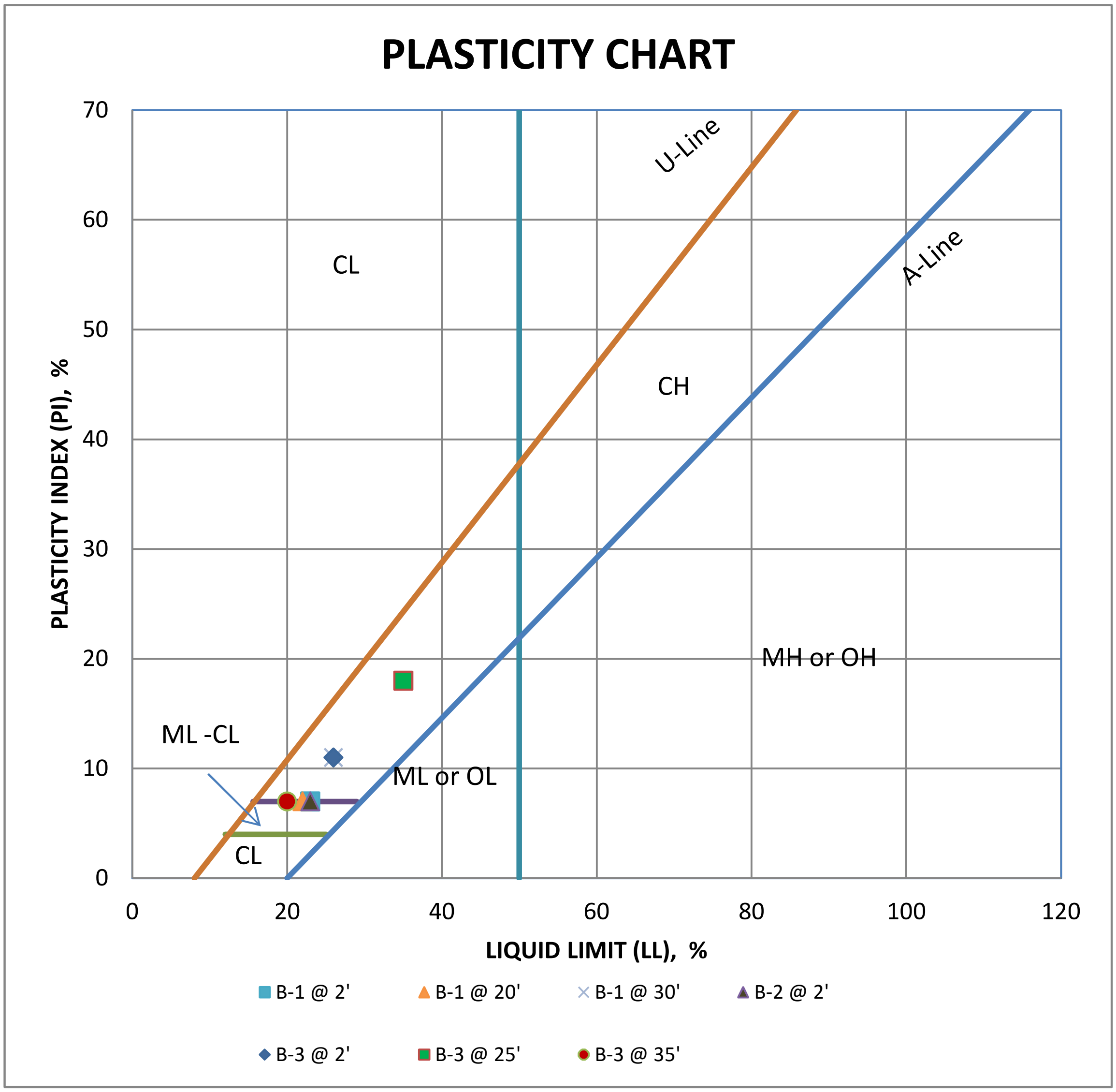
550 GATEWAY BLVD.
S. SAN FRANCISCO, CA

Date: February, 2016

Plate 7



SEIVE ANALYSIS 850 Gateway BLVD. S. San Francisco, CA	GeoEngineering Consultants
Date: February, 2016	Plate 8



PLASTICITY CHART

850 Gateway BLVD.

S. San Francisco, CA

Date: February, 2016	Plate 9

APPENDIX B

The Grading Specifications

Guide Specifications for Rock Under Floor Slabs

THE GRADING SPECIFICATIONS

1. General Description

1.1 These specifications have been prepared for the grading and site development of the subject residential development. GEC, hereinafter described as the Soil Engineer, should be consulted prior to any site work connected with site development to ensure compliance with these specifications.

1.2 The Soil Engineer should be notified at least two working days prior to any site clearing or grading operations on the property in order to observe the stripping of organically contaminated material and to coordinate the work with the grading contractor in the field.

1.3 This item shall consist of all clearing or grubbing, preparation of land to be filled, filling of the land, spreading, compaction and control of fill, and all subsidiary work necessary to complete the grading of the filled areas to conform with the lines, grades, and slopes as shown on the accepted plans. The Soil Engineer is not responsible for determining line, grade elevations, or slope gradients. The property owner, or his representative, shall designate the person or organizations who will be responsible for these items of work.

1.4 The contents of these specifications shall be integrated with the soil report of which they are a part, therefore, they shall not be used as a self-contained document.

2. Tests

The standard test used to define maximum densities of all compaction work shall be the ASTM D1557-91 Laboratory Test Procedure. All densities shall be expressed as a relative compaction in terms of the maximum dry density obtained in the laboratory by the foregoing standard procedure.

3. Clearing, Grubbing, and Preparing Areas To Be Filled

3.1 If encountered, all vegetable matter, trees, root systems, shrubs, debris, and organic topsoil shall be removed from all structural areas and areas to receive fill.

3.2 If encountered, any soil deemed soft or unsuitable by the Soil Engineer shall be removed. Any existing debris or excessively wet soils shall be excavated and removed as required by the Soil Engineer during grading.

3.3 All underground structures shall be removed from the site such as old foundations, abandoned pipe lines, septic tanks, and leach fields.

3.4 The final stripped excavation shall be approved by the Soil Engineer during construction and before further grading is started.

3.5 After the site has been cleared, stripped, excavated to the surface designated to receive fill, and scarified, it shall be bladed until it is uniform and free from large clods. The native subgrade soils shall be moisture conditioned and compacted to the requirements as specified in the grading section of this report. Fill can then be placed to provide the desired finished grades. The contractor shall obtain the Soil Engineer's approval of subgrade compaction before any fill is placed.

4. Materials

4.1 All fill material shall be approved by the Soil Engineer. The material shall be a soil or soil-rock mixture which is free from organic matter or other deleterious substances. The fill material shall not contain rocks or lumps over 6 inches in greatest dimension and not more than 15% larger than 2-1/2 inches. Materials from the site below the stripping depth are suitable for use in fills provided the above requirements are met.

4.2 Materials existing on the site are suitable for use as compacted engineered fill after the removal of all debris and organic material. All fill soils shall be approved by the Soil Engineer in the field.

4.3 Should import material be required, it must meet the specifications as delineated in the body of this report.

5. Placing, Spreading, and Compacting Fill Material

5.1 The fill materials shall be placed in uniform lifts of not more than 8 inches in uncompacted thickness. Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to obtain uniformity of material in each layer. Before compaction begins, the fill shall be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with water if it is too dry.

5.2 After each layer has been placed, mixed, and spread evenly, either import material or native material shall be compacted to a relative compaction designated for engineered fill.

5.3 Compaction shall be by footed rollers or other types of acceptable compacting rollers. Rollers shall be of such design that they will be able to compact the fill to the specified density. Rolling shall be accomplished while the fill material is within the specified moisture content range. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to ensure that the required density has been obtained. No ponding or jetting shall be permitted.

5.4 Field density tests shall be made in each compacted layer by the Soil Engineer in accordance with Laboratory Test Procedure ASTM D1556-64 or D2922-71. When footed rollers are used for compaction, the density tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements on any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

5.5 No soil shall be placed or compacted during periods of rain nor on ground which contains free water. Soil which has been soaked and wetted by rain or any other cause shall not be compacted until completely drained and until the moisture content is within the limits hereinbefore described or approved by the Soil Engineer. Approval by the Soil Engineer shall be obtained prior to continuing the grading operations.

6. Pavement

6.1 The proposed subgrade under pavement sections, native soil, and/or fill shall be compacted to a minimum relative compaction of 95% at 3% above optimum moisture content for a depth of 6 inches.

6.2 All aggregate base material placed subsequently should also be compacted to a minimum relative compaction of 95% based on the ASTM Test Procedure D1557-91. The construction of the pavement in the parking and traffic areas should conform to the requirements set forth by the latest Standard Specifications of the Department of Transportation of the State of California and/or City of South San Francisco, Department of Public Works.

6.3 It is recommended that soils at the proposed subgrade level be tested for a pavement design after the preliminary grading is completed and the soils at the site design subgrade levels are known.

7. Utility Trench Backfill

7.1 The utility trenches extending under concrete slabs-on-grade shall be backfilled with native on-site soils or approved import materials and compacted to the requirements pertaining to the adjacent soil. No ponding or jetting will be permitted.

7.2 Utility trenches extending under all pavement areas shall be backfilled with native or approved import material and properly compacted to meet the requirements set forth by the City of South San Francisco, Department of Public Works.

7.3 Where any opening is made under or through the perimeter foundations for such items as utility lines and trenches, the openings must be resealed so that they are watertight to prevent the possible entrance of outside irrigation or rain water into the underneath portion of the structures.

8. Subsurface Line Removal

8.1 The methods of removal will be designated by the Soil Engineer in the field depending on the depth and location of the line. One of the following methods will be used.

8.2 Remove the pipe and fill and compact the soil in the trench according to the applicable portions of sections pertaining to compaction and utility backfill.

8.3 The pipe shall be crushed in the trench. The trench shall then be filled and compacted according to the applicable portions of Section 5.

8.4 Cap the ends of the line with concrete to prevent entrance of water. The length of the cap shall not be less than 5 feet. The concrete mix shall have a minimum shrinkage.

9. Unusual Conditions

9.1 In the event that any unusual conditions not covered by the special provisions are encountered during the grading operations, the Soil Engineer shall be immediately notified for additional recommendations.

10. General Requirements

10.1 The contractor shall conduct all grading operations in such a manner as to preclude wind blown dirt and dust and related damage to neighboring properties. The means of dust control shall be left to the discretion of the contractor and he shall assume liability for claims related to wind blown material.

GUIDE SPECIFICATIONS FOR ROCK UNDER FLOOR SLABS

Definition

Graded gravel or crushed rock for use under slabs-on-grade shall consist of a minimum thickness of mineral aggregate placed in accordance with these specifications and in conformance with the dimensions shown on the plans. The minimum thickness is specified in the accompanying report.

Material

The mineral aggregate shall consist of broken stone, crushed or uncrushed gravel, quarry waste, or a combination thereof. The aggregate shall be free from deleterious substances. It shall be of such quality that the absorption of water in a saturated dry condition does not exceed 3% of the oven dry weight of the sample.

Gradation

The mineral aggregate shall be of such size that the percentage composition by dry weight, as determined by laboratory sieves (U.S. Sieves) will conform to the following gradation:

Sieve Size	Percentage Passing
3/4"	90-100
No. 4	25-40
No. 8	18-33
No. 200	0-3

Placing

Subgrade, upon which gravel or crushed rock is to be placed, shall be prepared as outlined in the accompanying soil report.