

APPENDIX G

GEOTECHNICAL EXPLORATION REPORT

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**1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA**

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Kristen Gates
Hanover R.S. Limited Partnership
5847 San Felipe, Suite 3600
Houston, TX 77057

PREPARED BY
ENGEO Incorporated

July 27, 2018

PROJECT NO.
14140.000.000

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Ms. Kristen Gates
Hanover R.S. Limited Partnership
5847 San Felipe, Suite 3600
Houston, TX 77057

Subject: 1095 Rollins Road
Burlingame, California

GEOTECHNICAL EXPLORATION

Dear Ms. Gates:

ENGEO prepared this geotechnical report for Hanover R.S. Limited Partnership as outlined in our agreement dated July 19, 2017. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated



Todd Bradford, PE
tb/jjt/jf



Josef J. Tootle, GE

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design of the proposed mixed-use development in Burlingame, California. We prepared this report as outlined in our agreement dated July 19, 2017. Hanover R.S. Limited Partnership authorized ENGEO to conduct the following scope of services:

- Review available literature and geologic maps
- Subsurface field exploration
- Soil laboratory testing
- Data analysis and conclusions
- Report preparation

This report was prepared for the exclusive use of our client and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The approximately 1.1-acre project site is located at 1095 Rollins Road in Burlingame California. The rectangular-shaped project site is bound to the north by Rollins Road, to the east and south by multi-story apartment buildings, and to the west by a gas station. The San Francisco Bay shoreline is located approximately 1,200 feet to the north of the site. The site is currently occupied by a restaurant building and an elevated tennis court.

1.3 PROJECT DESCRIPTION

Based on our review of the provided information, we understand the project will consist of a podium structure with one level of below-grade parking and up to six levels of mixed-use above-grade construction. Various areas within the one-level basement will be deepened an additional level to accommodate mechanical car-stackers. We understand from speaking with you that you intend to excavate the entire site to the depth of the car stacker pits, construct your foundation elements, then backfill where needed to achieve the design basement elevation. Construction of underground utilities, retaining walls, flatwork, and landscaping will also be part of the proposed development.

Structural loads and grading plans were not available for our review at the time of our exploration. We assume that structural loads will be representative for this type of construction. We assume from the review of the floor plan that the basement level of parking and car stacker pits will require approximately 10 feet and 20 feet of cut, respectively.

2.0 FINDINGS

2.1 FIELD EXPLORATION

Our field exploration included drilling three borings and advancing three cone penetration tests (CPTs) at various locations on the site. We performed our field exploration between June 12 and June 13, 2018. We established the exploration locations by visual sighting from existing features and should be considered accurate only to the degree implied by the method used.

We observed drilling of the three borings at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted drill rig and crew to advance the borings using 4-inch-diameter rotary wash methods. The borings were advanced to depths ranging from approximately 60 to 70 feet below ground surface (bgs). We permitted and backfilled the borings in accordance with the requirements of San Mateo County Environmental Health.

We obtained soil samples at various intervals using standard penetration test (SPT) samplers with a 2-inch-outside-diameter (O.D. split-spoon sampler) and California Modified samplers with 2½-inch-inside-diameter (I.D.). We obtained the blow counts shown on our bore logs with an automatic trip, 140-pound hammer with a 30-inch free fall. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

Additionally, we retained the services of a subcontractor with a CPT rig to advance three cone penetration tests to depths of up to 80 feet below ground surface (bgs) in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the surface sleeve (F_s), and pore pressure (U) (Robertson and Campanella, 1988). Pore pressure dissipation tests were performed to measure the approximate subsurface phreatic surface.

We provide additional information about specific subsurface conditions at each location in our boring logs in Appendix A and CPT interpretations in Appendix B. The soil type, color, consistency, and visual classification provided in the logs are in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Geology

The site is located in the Coast Ranges geomorphic province of California. The Coast Ranges are dominated by a series of northwest-trending ridges and valleys formed by faulting and folding of the earth's crust. As shown on the Geologic Map prepared by Pampeyan (1994), artificial fill (af) is mapped on the site (Figure 3). Historical development of the San Francisco Bay shoreline resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands, tributaries, and creek beds in an effort to reclaim land (Nichols and Wright, 1971). Historical mapping of the area shows the project site to be located within a former tidal marsh that was subsequently filled during development of the area.

2.2.2 Seismicity

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future. The distance and maximum anticipated moment magnitude of the nearest active faults are listed in Table 2.2.2-1. In addition, Figure 5 shows the approximate locations of these faults and significant historic earthquakes recorded within the San Francisco Bay Region. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Bryant and Hart, 2007).

TABLE 2.2.2-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

FAULT NAME	DISTANCE FROM SITE (MILES)	MAXIMUM MOMENT MAGNITUDE
San Andreas	2.3	8.1
San Gregorio Connected	8.9	7.5
Monte Vista-Shannon	12.0	6.5
Hayward-Rodgers Creek	15.9	7.3
Calaveras	24.4	7.0
Mount Diablo Thrust	26.5	6.7
Green Valley Connected	29.5	6.8
Greenville Connected	35.2	7.0

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated.

As shown in Figure 4, the site is located within a State of California Seismic Hazard Zone (CGS, 2016) for areas that may be susceptible to liquefaction. Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium-dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable.

2.3 SITE HISTORY

We reviewed individual aerial photographs of the site dated 1946, 1956, 1968, 1980, 1987, 1993, 2000, 2002, 2005, 2009, 2010, 2012, and 2014 and a topographic map dated 1896 that were available online at <http://www.historicaerials.com>. Based on our review, the property was previously situated on the border of marshland and the historic San Francisco Bay shoreline. Efforts to extend the shoreline further into the bay through the placement of fill material are evident in the 1946 photography. The site appears to have been completely filled by 1956 while the existing restaurant building and elevated tennis court structure appear by 1980. The site has remained relatively unchanged since 1980.

2.4 SURFACE CONDITIONS

The project site appears relatively flat and is currently occupied by a restaurant building, an elevated tennis court structure, and minor landscaping.

2.5 SUBSURFACE CONDITIONS

The site is underlain by between 5 and 15 feet of undocumented fill consisting of sandy lean clay, clayey sand, gravelly clay, and fat clay with sand. Beneath the undocumented fill, we encountered an approximately 10-foot-thick layer of loose to medium-dense clayey sand. Following the sand deposit, we generally encountered lean clay, sandy clay, and fat clay to the bottom of our explorations. We also observed an approximately 5-foot-thick layer of high clay content medium dense sand at around 30 to 35 feet bgs. We also noted a lens of dense clayey sand at 55 feet bgs in Borings 1-B2 and 1-B3

Consult Figure 2 and boring logs (Appendix A) for specific subsurface conditions at each location.

2.6 GROUNDWATER CONDITIONS

During our CPT exploration, we performed pore pressure dissipation tests that indicated a groundwater level approximately 13 feet bgs. However, we performed environmental sampling during the preparation of a Phase II Environmental Site Assessment (ENGEO, 2018) where we observed groundwater at approximately 5 feet bgs. Additionally, historical high groundwater level in the area is mapped by the California Geological survey at a depth of less than 10 feet bgs (Figure 6).

We recommend that the design groundwater elevation of the project be assumed to be 5 feet bgs. Fluctuations in the level of groundwater may occur due to variations in rainfall, tidal fluctuation, irrigation practice, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

We performed laboratory tests on select soil samples to evaluate their engineering properties. For this project, we performed laboratory testing as shown in the table below.

TABLE 2.7-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD
Natural Unit Weight	ASTM D7263
Natural Moisture Content	ASTM D2216
Plasticity Index (Wet Method)	ASTM D4318
Grain Size Distribution	ASTM D6913
Percent Passing #200 Sieve	ASTM D1140
Triaxial Compression – Unconsolidated, Undrained (TXUU)	ASTM D2850
Unconfined Compression	ASTM D2166

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the proposed project may be designed as planned, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical considerations influencing the project are:

- The presence of undocumented fill.
- Shallow groundwater.
- Liquefaction-induced settlement.

3.1 UNDOCUMENTED FILL

During our investigation, we encountered fill extending between 7 and 15 feet below existing grade in all three of our borings. Based on the floor plan, the majority of undocumented fill will be removed for the excavation of the parking garage. Structures built at grade may be potentially influenced by the presence of undocumented fill. Non-engineered fill can undergo excessive settlement, especially under new fill or building loads. We present fill removal recommendations in Section 5.2. for the existing fill.

3.2 SHALLOW GROUNDWATER

Given the final design depth of the proposed excavation for the planned structure, groundwater will influence the excavation and the bottom of the excavation. As such, temporary construction dewatering, and design of below-grade portions of the structure for hydrostatic conditions and waterproofing are anticipated. Shallow groundwater can:

1. Delay grading activities, especially compacting soil below basement elevations.
2. Require construction dewatering.
3. Cause moisture damage to sensitive floor coverings.
4. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.
5. Require waterproofing for the proposed basement structures.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and liquefaction induced damages. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, lateral spreading, and landslides is considered low to negligible at the site.

The Association of Bay Area Governments does not map the project site within a potential tsunami inundation zone. Given the proximity to the San Francisco Bay and low elevation, you should consult your civil engineer regarding flooding potential.

3.3.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the 2007 California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction/Cyclic Softening

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable.

We performed liquefaction analysis using the computer software CLiq Version 2.0 developed by GeoLogismiki. We performed our analysis based on procedures by Robertson (2009). We estimated the Cyclic Stress Ratio (CSR) using a MCE Geometric Mean Peak Ground Acceleration (PGA_M) value of 0.82g as outlined in the CBC and moment magnitude of 8.1 based on the characteristic magnitude of the nearby San Andreas fault. We evaluated the liquefaction potential for the soil encountered below a groundwater depth of 5 feet bgs. The results of our analyses indicate there are some minor subsurface layers potentially susceptible to liquefaction.

3.3.3.1 Liquefaction-Induced Settlement

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface. Densification occurs as excess pore pressures dissipate, resulting in vertical settlement at the ground surface.

We calculated potential liquefaction-induced settlement estimates using the program Clq for soils in the top 50 feet of the site. The procedures used in Clq are based on the methods published by Zhang, G., Robertson, P.K., and Brachman, R. (2002).

Additionally, we performed laboratory testing on some of the transitional silts and clays that showed as potentially liquefiable in the CPT analysis. Based on fine-grained liquefaction triggering screening procedures outlined in Bray and Sancio (2006), the silt and clay beginning at approximately 40 feet bgs is not susceptible to liquefaction.

We estimate approximately 1 inch of liquefaction-induced ground surface settlement and approximately ½ inch of liquefaction-induced building settlement. We estimate that differential

settlement over a span of 50 feet during a seismic event will be approximately half of the total settlement.

3.3.3.2 Surface Manifestation

We also evaluated the capping effect of overlying non-liquefiable soil, based on guidelines provided by Ishihara (1985) and Youd and Garriss (1995). For liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures.

We assessed conditions assuming that future grades will approximately match the existing grades. Based on our analysis, the project site surrounding the building is likely to exhibit surface manifestations of liquefied material in the form of sand boils and fissures.

3.4 SOIL CORROSION POTENTIAL

As part of this study, we collected three soil samples from various areas across the site and at varying depths. The samples were combined and then sent to CERCO Analytical, who performed testing according to ASTM Test Methods for pH, resistivity, sulfate, and chloride. At the time of this publication, the test results were not available for review. We will provide the results and evaluation under separate cover at a later date.

3.5 2016 CBC SEISMIC DESIGN PARAMETERS

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, and the anticipated basement excavation removals, we characterized the site as Site Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.5-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.5-1: 2016 CBC Seismic Design Parameters, Latitude: 37.3337 Longitude: -121.90628

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.11
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	1.00
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.11
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.50
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.41
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	1.00
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.50
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.82
Long period transition-period, T_L	12 sec

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.
2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Excavation, shoring, and earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 SITE RECOMMENDATIONS

5.1 DEMOLITION AND STRIPPING

After demolition of the existing buildings, the site should be cleared of all obstructions, including existing foundations and debris that are not cleared by the excavation of the proposed parking garage. Any existing underground utilities at the site should be identified and either properly abandoned or relocated. Holes resulting from the removal of underground obstructions extending below the proposed finish grades should be cleared and backfilled with suitable properly compacted fill.

Areas containing surface vegetation or organic-laden topsoil should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by the geotechnical engineer at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

5.2 NON-ENGINEERED FILL REMOVAL

As previously mentioned, we encountered up to 15 feet of non-engineered fill at the site. Such fill can undergo excessive and non-uniform settlement under new loads. We recommend removing all non-engineered fill within the building footprint and, if necessary, replacing it with moisture conditioned and compacted fill. In areas to receive other improvements, including site retaining walls, secondary slabs on grade, or flatwork we recommend removing at least 5 feet of non-engineered fill and replacing it with moisture conditioned and compacted fill. We should have an ENGEO representative onsite to determine if the 5 feet of removal is sufficient to create a stable base.

6.0 FOUNDATION RECOMMENDATIONS

Based on the available soil data and anticipated structure type, the proposed structure can be supported on a mat foundation. The major considerations in foundation design at the site are the building loads, hydrostatic pressures, and potential uplift forces. Generally, a mat foundation will help bridge areas of localized settlement. Due to shallow groundwater and depth of excavation, the mat foundation should be designed for uplift forces.

6.1 MAT FOUNDATIONS

The structure may be supported on a rigid mat foundation. The thickness of the mat will be driven by the structural design. Mat foundations should be thick enough to resist hydrostatic uplift pressures.

Provided the remedial grading recommendations provided in Section 6.1.3 are incorporated during construction, the structural mat may be designed to impose an average allowable bearing pressure of at most 4,500 pounds per square foot (psf) for dead-plus-live loads. The allowable bearing capacity may be increased to 5,500 psf in areas of loading concentration. These values may be increased by one-third when considering transient loads, such as wind or seismic. If a spring constant is needed for design, a modulus of subgrade reaction (k_s) of 100 pounds per square inch per inch of deflection (psi/in) can be used with a maximum allowable pressure of 4,000 psf.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soil and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.30 can be used between concrete and the subgrade. An equivalent fluid unit weight of 250 pcf may be used for passive pressure.

6.1.1 Settlement

We recommend mat foundation design consider the following settlement considerations:

TABLE 6.1.1-1: Settlement Considerations

TOTAL SEISMICALLY INDUCED SETTLEMENT (INCHES)	TOTAL STATIC SETTLEMENT (INCHES)
1/2	1 1/4

We anticipate the weight of the soil being excavated will offset a portion of the proposed dead loads of the structure. As shown in the current floor plans, we anticipate that static settlement below the bottom of the mat will be approximately 1 1/4 inch under allowable bearing pressures. Differential settlement over a distance of 50 feet will be approximately half the total settlement. Most of the static settlement will occur during construction.

6.1.2 Uplift Forces

We anticipate that the basement-level garage will be below the groundwater level and will have to be designed for hydrostatic uplift loads. Uplift resistance can be provided by the weight of the foundation elements and structural loads. Additional resistance to uplift may be provided by

installing hold-down anchors, if necessary. However, given the intermittent layers of liquefiable material, the anchors would need to be extended to a depth that is likely cost prohibitive.

For planning purposes, the anchor capacity should be evaluated using an ultimate bond strength between grout and soil of 1,400 psf. This value assumes the grout is placed under pressure during casing withdrawal and may be increased by 30 percent for wind and seismic loading. Due to the intermittent potentially liquefiable layers underlying the site, the adhesion between grout and the soil should begin at 30 feet bgs. The anchors should be spaced no closer than 3 times the shaft diameter and have a minimum embedment length of 50 feet bgs. The design-build contractor should design based on an appropriate bond capacity and this value should be verified in the field based on load testing. We should review appropriate design documentation to assure that our geotechnical recommendations are being followed by the anchor designer.

6.1.3 Subgrade Treatment for Structural Mat Foundations

The mat foundation should be placed neat against the undisturbed soil. The pad subgrade should not be allowed to dry before placing concrete. The pad subgrade should be checked by a representative of our firm prior to concrete placement for compliance with these moisture requirements and to confirm the adequacy of the bearing soil. Soft or loose soil present at the bottom of the excavation should be removed and replaced with engineered fill or lean concrete. To reduce the disturbance of the mat subgrade once prepared, we recommend that the subgrade be covered with a “rat” or “mud” slab of lean concrete at least 2 inches thick.

6.2 WATERPROOFING

We recommend that the design groundwater level for the project be assumed to be at a depth of 10 feet bgs. Due to operational concerns, potential settlement impacts on nearby buildings and agency permissions, we recommend that permanent dewatering not be considered, and recommend that the concrete slabs and walls that will be constructed below the design groundwater elevation be waterproofed and designed to resist hydrostatic and/or uplift pressures. The waterproofing should be designed by a consultant that specializes in permanent waterproofing construction.

7.0 BASEMENT WALLS AND NON-BUILDING RETAINING WALLS

7.1 BASEMENT WALL LATERAL SOIL PRESSURES

The garage basement walls will act as retaining walls. Basement walls should be designed for at-rest lateral loading conditions. We recommend placing a drain behind all walls above the design groundwater level to reduce hydrostatic pressure; if a drain is not feasible, hydrostatic pressure should be added to the equivalent fluid pressure. Recommendations for wall drainage follow in a later section. Some miscellaneous cantilever retaining walls may be required and can be designed for active lateral loading conditions. The recommended lateral equivalent fluid pressures (static case) are presented below:

TABLE 7.1-1: Lateral Earth Pressures

LOADING CONDITION	DRAINED PRESSURE (PCF)	UNDRAINED PRESSURE (PCF)
Cantilevered (Active)	45	80
Restrained (At-Rest)	65	90

The design groundwater level should be assumed to be located at approximately 5 feet below ground surface. Permanent dewatering is not recommended; therefore, basement walls should be designed to resist hydrostatic pressures.

Where surcharge loads from vehicles are expected within a distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 100 psf to be applied over the entire height of the wall or 10 feet, whichever is less.

7.2 SEISMIC DESIGN CONSIDERATIONS

Seismic conditions need to be considered in the design of the garage basement retaining walls. Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures, and can be calculated as follows:

$$\Delta P = 15 \times H^2$$

Where H is the design height of the wall (in feet) and ΔP is the active incremental seismic force in pounds per foot of wall. This force has a horizontal direction and should be applied at $0.3 \times H$ from the base of the wall. This force should be combined with the appropriate active equivalent pressure, regardless if the wall is cantilevered or restrained.

7.3 NON-BUILDING RETAINING WALL LATERAL SOIL PRESSURES

Passive pressures acting on retaining walls may be assumed as 250 pounds per cubic foot (pcf), provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations unless it is confined by pavement or concrete slab. Active pressures may be taken from Table 7.1-1. Non-building retaining walls may be supported on continuous footings designed with an allowable bearing capacity of 1,500 psf.

7.3.1 Wall Drainage

Unless the non-building retaining walls are designed for hydrostatic pressures, these walls should be provided with drainage facilities. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about 1 foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper 1 foot of wall backfill should consist of clayey soils. Drainage should be collected by perforated pipes and directed a sump or, if feasible, drained into the storm drain system.

All backfill should be placed in accordance with recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls.

8.0 EXCAVATION DEWATERING AND SHORING

Excavation dewatering and shoring are temporary works that are typically the responsibility of the contractor to design, install, maintain and monitor. An experienced shoring and dewatering system designer should be retained to select and design these systems. The following sections

provide some general considerations that should be incorporated into shoring and dewatering system design. Geotechnical shoring design recommendations are dependent on performance criteria, the type of system selected and construction sequencing.

8.1 TEMPORARY SHORING

Temporary shoring pressures for the anticipated final excavation configuration are provided in Figure 6. Passive pressures shown may be applied over two pier diameters.

As information regarding construction equipment types and adjacent building loads become available, we can provide surcharge load recommendations to account for heavy construction equipment as well as the existing buildings adjacent the excavation. Additionally, once excavation and shoring sequencing is established, we will provide appropriate intermediate pressure diagrams to address the various stages of construction.

Given the proposed excavation depth, it may be necessary to restrain the shoring by using a single-level or multi-level system of tie-back anchors or to provide internal bracing. Prior to tie-back design and construction, permission from the neighboring properties will have to be obtained if tie-backs are to encroach into those adjacent properties. Tie-back anchors may be installed to avoid adjacent underground utilities. For planning purposes, the anchor capacity should be evaluated using an ultimate bond strength between grout and soil of 1,400 psf. The tiebacks may be installed through the selected shoring system with 15 to 20 degree inclinations.

8.2 CONSTRUCTION DEWATERING

There are a number of variables that will influence the effectiveness of a dewatering system, including the number, depth, screened interval, and pumping rate of wells. The local sewer agency may prohibit the discharge of groundwater into the system or may charge a fee to do so.

The water level should be maintained at least 3 feet below the bottom of the deepest excavation during construction. The selection of equipment, actual depth, and spacing of the wells should be determined by the dewatering designer/contractor. The dewatering system implemented should be selected to have minimal impact on the groundwater level surrounding the proposed excavation.

8.3 PRE-CONSTRUCTION SURVEY AND CONSTRUCTION MONITORING

Excavation construction will take place adjacent to existing buildings, streets, improvements, and underground utilities. We recommend that a pre-construction survey (e.g. crack survey) and monitoring program for the surrounding culverts, buildings, roadways, utilities, etc. that may be affected by construction activities be performed before and during construction. This survey will form a basis for any damage claims and also assist the contractor in assessing the performance of the shoring or excavation slopes. The pre-construction survey should record the elevation and horizontal position of critical installations within 50 feet minimum and may consist of photographs, video tapes, topographic survey, etc.

We also recommend that a system of construction monitoring be installed. This may consist of inclinometers and groundwater monitoring wells that are installed within a distance of 5 to 15 feet from the excavation towards the existing buildings. Vibration monitoring should be considered during operations of heavy equipment such as pile driving, demolition, etc. In addition, a

settlement survey should initially be performed on a weekly basis during excavation and on a monthly basis, approximately one month after the excavation has been completed, at a minimum.

9.0 SECONDARY SLAB-ON-GRADE

Secondary slabs include exterior walkways, patios, and steps. Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of the exterior flatwork is normal as it is part of the concrete curing process and should be expected. Frequent control joints should be provided during slab construction for control of cracking.

As a minimum requirement, slabs-on-grade should be reinforced with steel bars; in our experience, welded wire mesh may not be sufficient to control slab cracking. The Structural Engineer should design the actual slab reinforcement.

Exterior slabs should slope away from the building to prevent water from flowing toward the foundations. Consideration should be given to lightly moistening the site soils just prior to concrete placement.

9.1 SUBGRADE PREPARATION

Following demolition and stripping, all areas to receive minor fills, secondary slabs-on-grade, or pavements should be scarified to a depth of at least 12 inches, moisture conditioned and compacted to the requirements for engineered fill, presented below. The finished subgrade should be firm and non-yielding under the weight of compaction equipment.

9.2 SUITABLE FILL MATERIALS

Site soil containing less than 3 percent organics are suitable for use as engineered fill. With the exception of the high plastic clay noted in the northern portion of the site, the site material is suitable for reuse as fill.

The geotechnical engineer should be informed if any soil import is contemplated. A sample of the proposed import material should be submitted to the geotechnical engineer for evaluation prior to delivery at the site.

9.3 PLACEMENT OF FILL

After removal of any soft soils and loose fill, the exposed non-yielding surface of all areas to receive minor fills, secondary slabs-on-grade, or pavements should be scarified to a depth of 12 inches, moisture conditioned, and recompact to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts. The lift thickness should not exceed 12 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to onsite materials:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content.
Required Relative Compaction:	Minimum 90 percent

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material.

9.4 DRAINAGE

Improper drainage may result in fill saturation with consequent loss of compaction and fill strength. It is very important that the perimeter of the structure be positively graded at all times to provide for rapid removal of surface water. Ponding of water or seepage toward foundation systems at any time during or after construction must be prevented.

Ponding of stormwater must not be permitted on the building during prolonged periods of inclement weather. Stormwater from roof downspouts should be conveyed in closed drain systems to a solid pipe that discharges to the street or storm drain system.

10.0 PAVEMENT DESIGN

10.1 FLEXIBLE PAVEMENT

For design, we assumed a Resistance (R-Value) of 5. The design sections may be reduced based on R-Value testing of samples collected from actual pavement subgrade. Using a range of traffic indices, we developed the following recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 10.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	HOT MIX ASPHALT (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	10
6	3½	13
7	4	16
8	5	18

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

10.2 RIGID PAVEMENT

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- A minimum section of 6 inches of Portland Cement concrete over 6 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.

- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.
- As an additional measure to improve performance, rigid pavements may be constructed with thickened edges extending at least beneath the aggregate base and 2 inches into compacted soil to reduce water infiltration, where adjacent to landscaped areas.

10.3 AGGREGATE BASE COMPACTION

Aggregate base should be compacted at or above optimum moisture and a minimum of 95 percent relative compaction. Aggregate Base should meet the requirements for $\frac{3}{4}$ -inch maximum Class 2 Aggregate Base in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

11.0 UTILITIES

We anticipate that differential settlement between the building and surrounding unimproved areas could be up to approximately 1 inch. Entry and pipe connections should be assessed to confirm the anticipated differential settlement can be tolerated; otherwise, flexible entry and pipe connections may be considered.

We recommend that utility trench backfilling be done under the observation of a geotechnical engineer. Pipe zone backfill (i.e., material beneath and immediately surrounding the pipe) may consist of a well-graded import or native material less than $\frac{3}{4}$ inch in maximum dimension. Trench zone backfill (i.e., material placed between the pipe zone backfill and the ground surface) may consist of native soil compacted in accordance with recommendations for engineered fill.

Where import material is used for pipe zone backfill, we recommend it consist of fine- to medium-grained sand or a well-graded mixture of sand and gravel, and that this material not be used within 2 feet of finish grades. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of: (1) soil into the relatively large void spaces present in this type of material, and (2) water along trenches backfilled with this type of material. All utility trenches entering buildings and paved areas must be provided with an impervious seal consisting of native materials or concrete where the trenches pass under the building perimeter or curb lines. The impervious plug should extend at least 3 feet to either side of the crossing. This is to prevent surface water percolation into the sands under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with City of Burlingame requirements. Compaction of trench backfill by jetting is not an appropriate method of compaction. If there appears to be a conflict between the City or other agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

12.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 1095 Rollins Road project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include soil volume change factors or flood potential. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface

conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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FIGURES

FIGURE 1: Vicinity Map

FIGURE 2: Site Plan

FIGURE 3: Regional Geologic Map

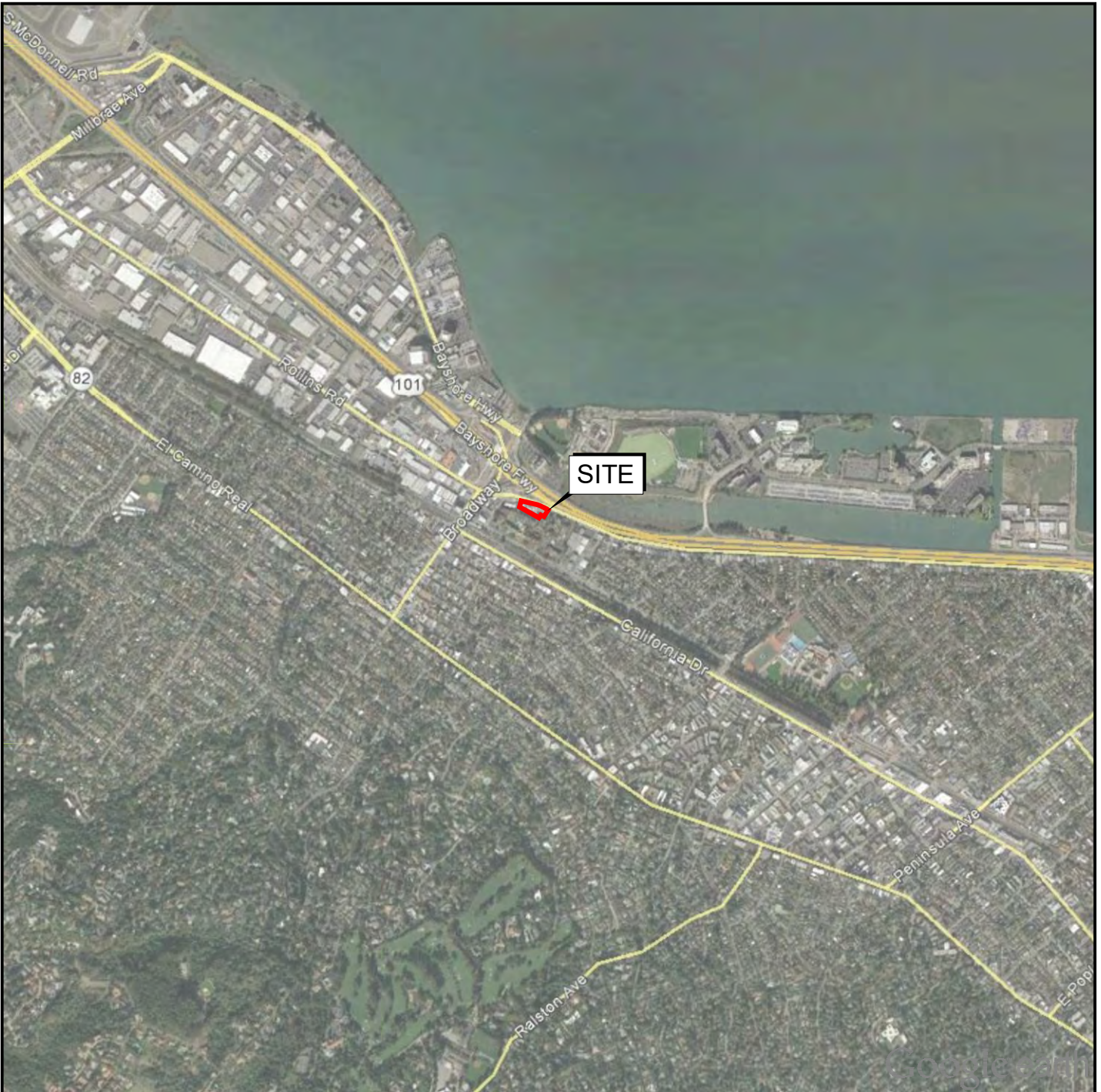
FIGURE 4: Seismic Hazard Zone Map

FIGURE 5: Regional Faulting and Seismicity Map

FIGURE 6: Regional Groundwater Map

FIGURE 7: Lateral Earth Pressure for Temporary Shoring

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BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE



VICINITY MAP
1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14110.000.000

SCALE: AS SHOWN

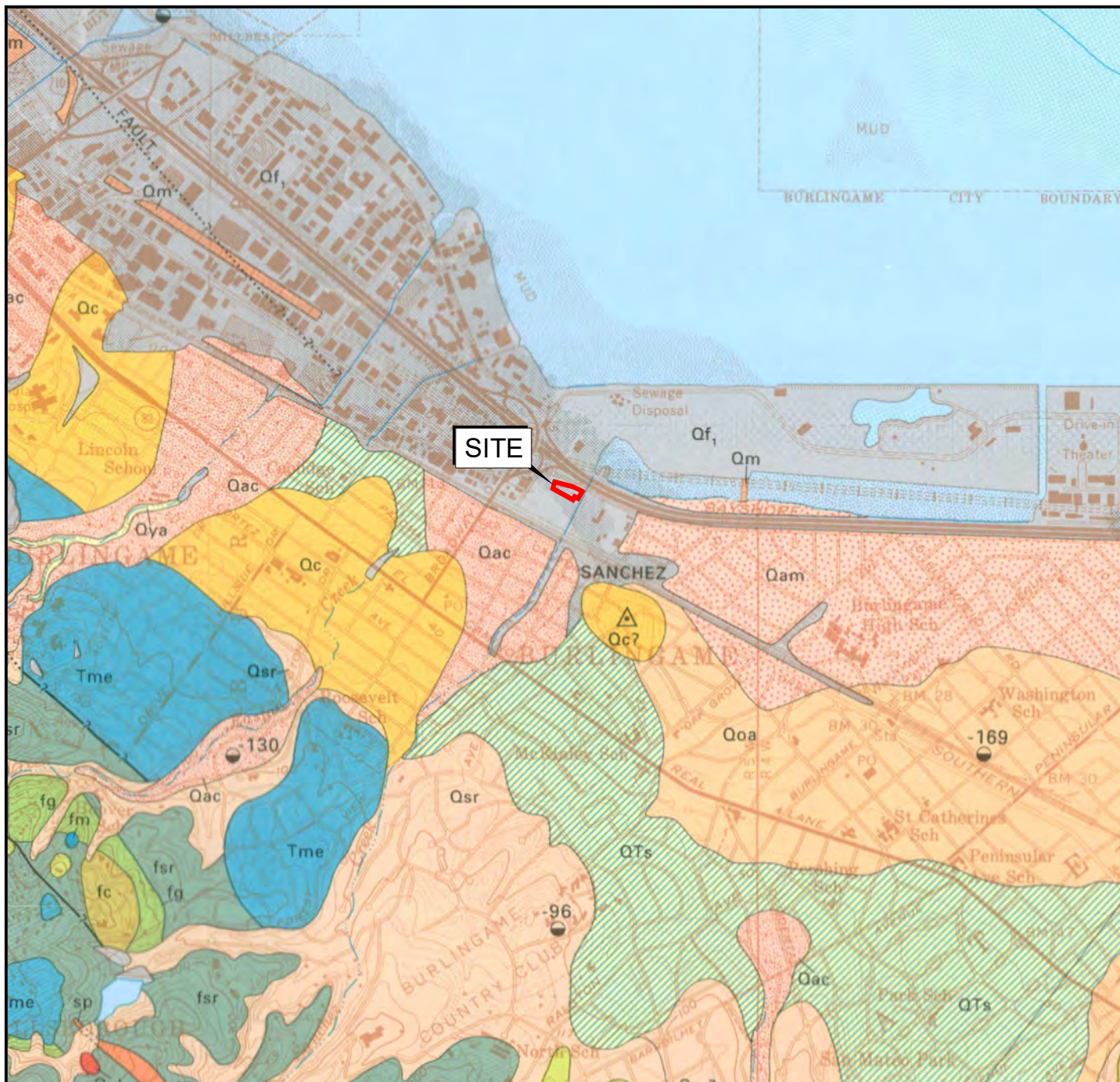
DRAWN BY: GLJ

CHECKED BY: TTB

FIGURE NO.

1





EXPLANATION

- GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED
- FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL.



- Qf1 ARTIFICIAL FILL
- QTs SEDIMENTARY DEPOSITS
- Qam MEDIUM-GRAINED ALLUVIUM (HOLOCENE)
- Qac COARSE-GRAINED ALLUVIUM (HOLOCENE)
- Qsr COLLUVIUM
- Qm BAY MUD
- Qoa OLDER ALLUVIUM (PLEISTOCENE)
- Qc COLMA FORMATION
- Tme MERCED FORMATION

BASE MAP SOURCE: PAMPEYAN, 1994



REGIONAL GEOLOGIC MAP
1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14110.000.000

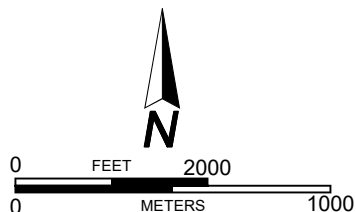
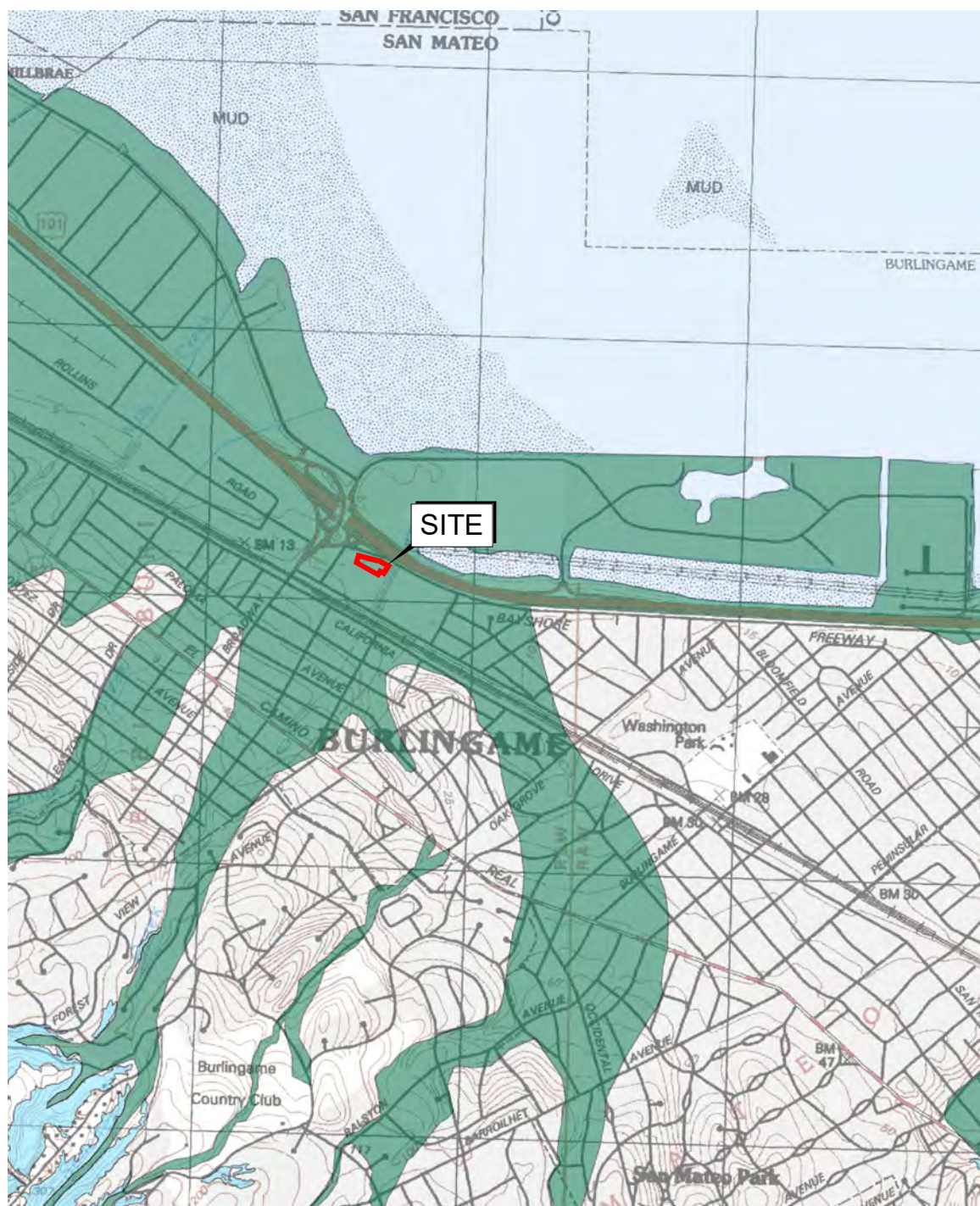
SCALE: AS SHOWN

DRAWN BY: GLJ

CHECKED BY: TTB

FIGURE NO.

3



EXPLANATION



LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006



SEISMIC HAZARD ZONE MAP
1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14110.000.000

SCALE: AS SHOWN

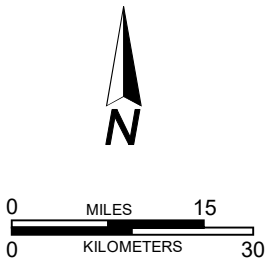
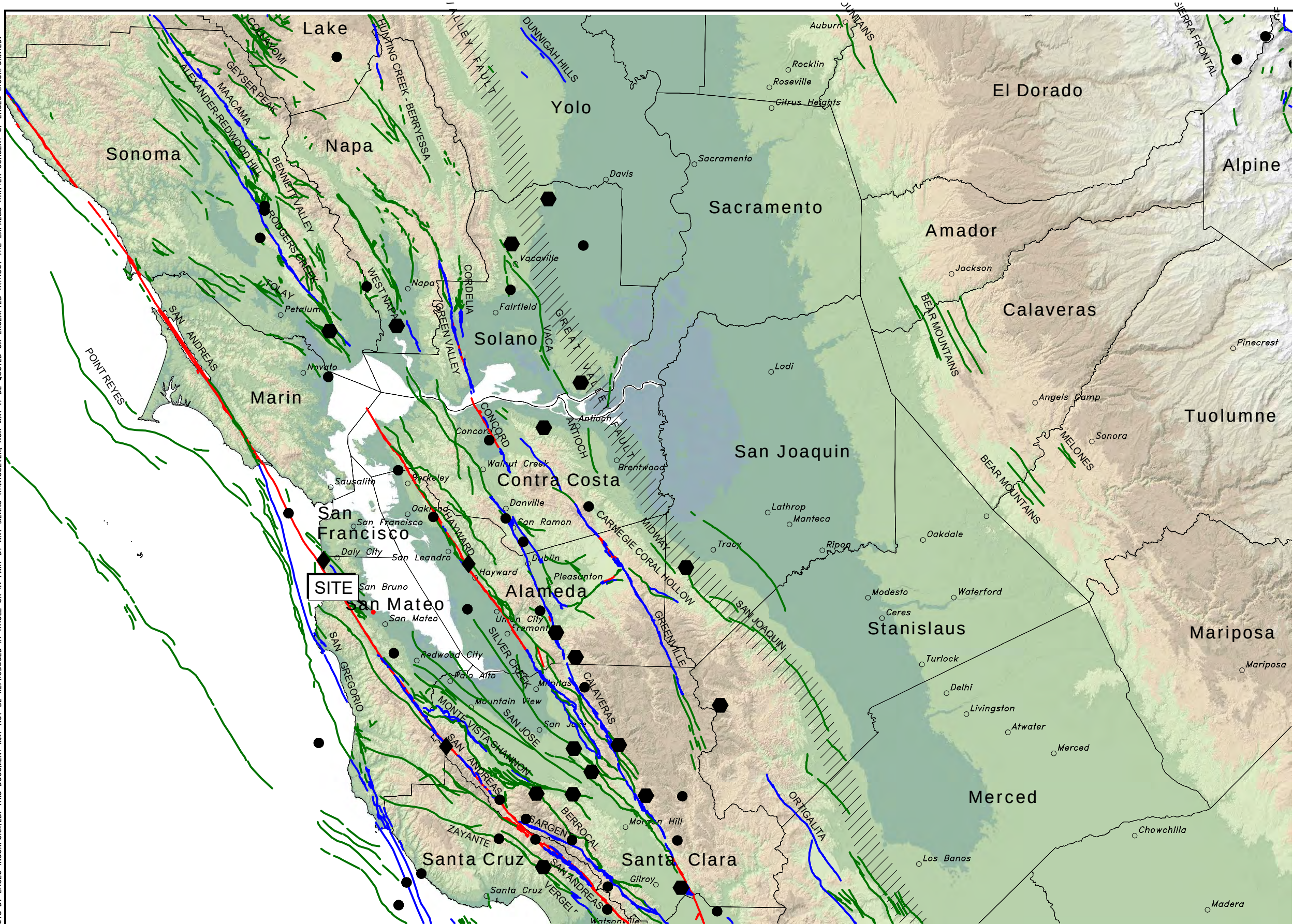
DRAWN BY: GLJ

CHECKED BY: TTB

FIGURE NO.

4

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EXPLANATION	
	MAGNITUDE 7+
	MAGNITUDE 6-7
	MAGNITUDE 5-6
	HISTORIC FAULT
	HOLOCENE FAULT
	QUATERNARY FAULT
	HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

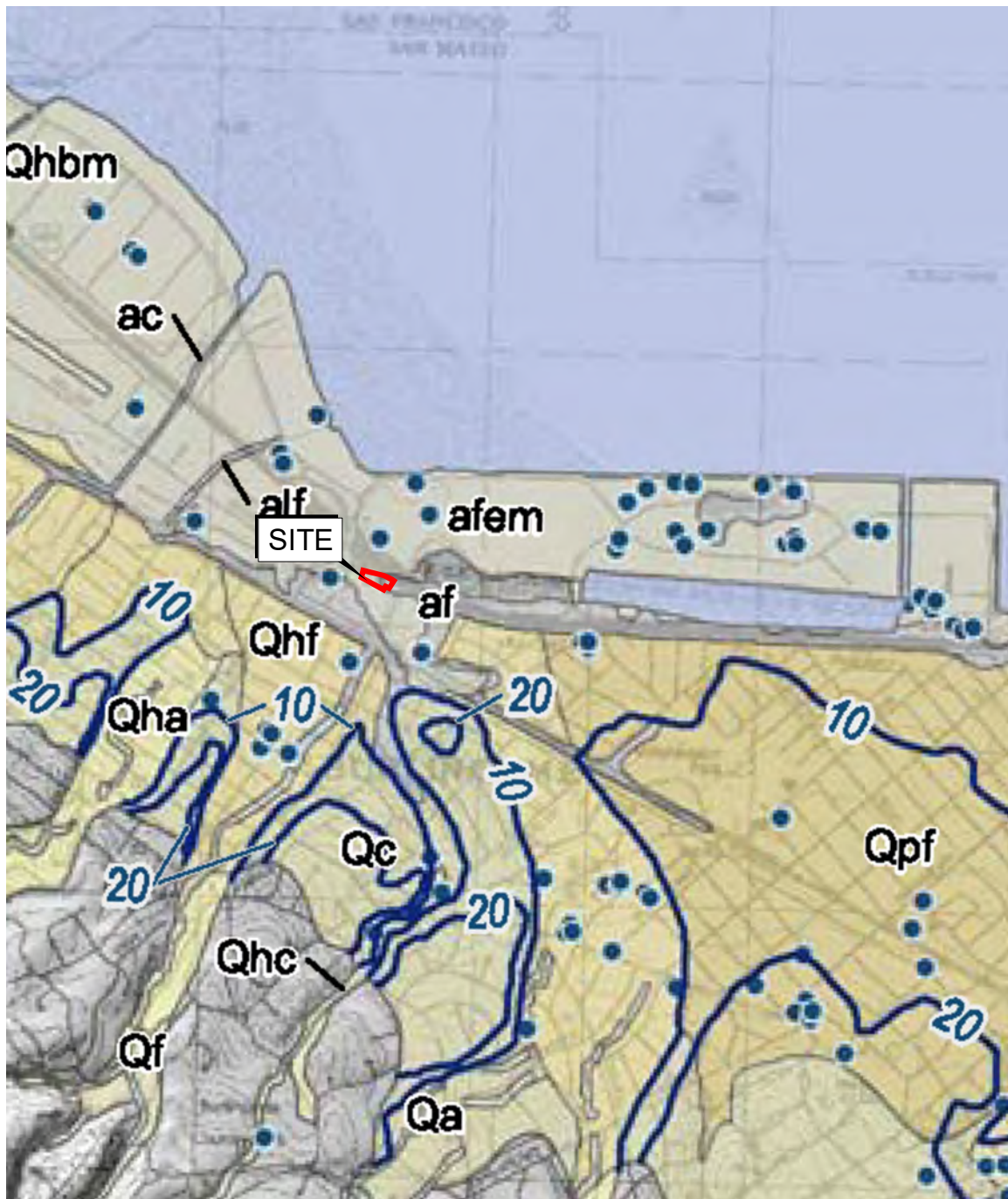


REGIONAL FAULTING AND SEISMICITY
1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14140.000.000
SCALE: AS SHOWN
DRAWN BY: GLJ CHECKED BY: TTB

FIGURE NO.
5

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION



GROUNDWATER CONTOUR

ac
afem
af
Qhbm

ARTIFICIAL STREAM CHANNELS
ARTIFICIAL FILL OVER ESTUARINE MUD
ARTIFICIAL UNDIFFERENTIATED FILL
SAN FRANCISCO BAY MUD

Qha UNDIFFERENTIATED ALLUVIUM (HOLOCENE)
Qhf ALLUVIAL FAN (HOLOCENE)
Qhc ALLUVIAL SEDIMENTS (LATE PLEISTOCENE TO HOLOCENE)
Qpf OLD ALLUVIAL FAN (PLEISTOCENE)
Qc COLMA FORMATION

0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: USGS



REGIONAL GROUNDWATER MAP
1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14110.000.000

SCALE: AS SHOWN

DRAWN BY: GLJ

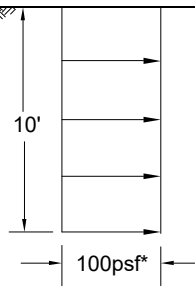
CHECKED BY: TTB

FIGURE NO.

6

G:\Drawing\DRAWING\3000 Plus\14140\000\000-GE-0618\1414000000-GE-7-SHOREDIA-0618.dwg Plot Date: 7-24-18 gjr/fe

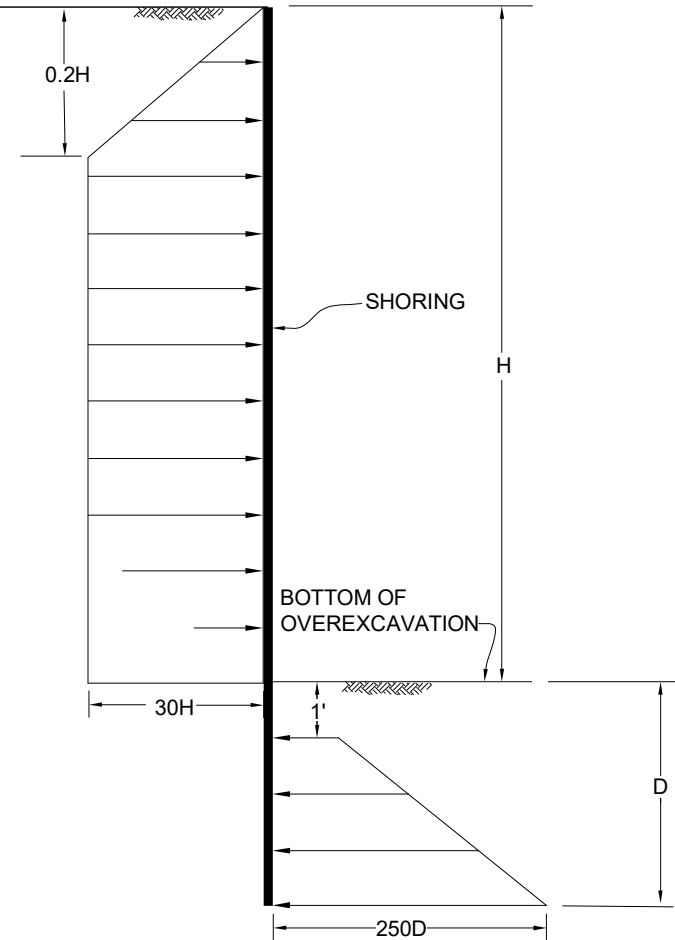
STREET TRAFFIC
SURCHARGE



*DESIGNER SHOULD CONFIRM THAT THIS VALUE IS ADEQUATE. (SHOULD BE INCREASED IF LARGE EQUIPMENT OR STOCKPILED SOIL IS LOCATED ADJACENT TO SHORING)

NOTES:

1. ASSUME GROUNDWATER WILL BE MAINTAINED A MINIMUM OF THREE FEET BELOW BOTTOM OF EXCAVATION.
2. D AND H ARE TO BE IN FEET.
3. SAFETY FACTOR TO BE INCLUDED BY DESIGNER.
4. ASSUMES LEVEL GROUND SURFACE AT TOP OF SHORING.
5. ASSUMES AT LEAST ONE LEVEL OF TIEBACKS OR INTERNAL BRACING.
6. D MUST BE ADEQUATE TO PROVIDE STABILITY TO THE BOTTOM OF THE EXCAVATION AND PREVENT PIPING OF WATER.
7. SURCHARGE LOADS FROM ADJACENT BUILDINGS AND IMPROVEMENTS NOT INCLUDED.



LATERAL EARTH PRESSURE FOR TEMPORARY SHORING

1095 ROLLINS ROAD
BURLINGAME, CALIFORNIA

PROJECT NO.: 14140.000.000

SCALE: NO SCALE

DRAWN BY: GLJ

CHECKED BY: TTB

FIGURE NO.

7

ORIGINAL FIGURE PRINTED IN COLOR



APPENDIX A

BORING LOG KEY EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES

DESCRIPTION

COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES		GW - Well graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES		GP - Poorly graded gravels or gravel-sand mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES		GM - Silty gravels, gravel-sand and silt mixtures
		SANDS WITH OVER 12 % FINES		GC - Clayey gravels, gravel-sand and clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS			SW - Well graded sands, or gravelly sand mixtures
				SP - Poorly graded sands or gravelly sand mixtures
				SM - Silty sand, sand-silt mixtures
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %			SC - Clayey sand, sand-clay mixtures
				ML - Inorganic silt with low to medium plasticity
				CL - Inorganic clay with low to medium plasticity
	HIGHLY ORGANIC SOILS			OL - Low plasticity organic silts and clays
				MH - Elastic silt with high plasticity
				CH - Fat clay with high plasticity
				OH - Highly plastic organic silts and clays
				PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE

CLEAR SQUARE SIEVE OPENINGS

200 40 10 4 3/4" 3" 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS

VERY LOOSE
LOOSE
MEDIUM DENSE
DENSE
VERY DENSE

BLOWS/FOOT
(S.P.T.)
0-4
4-10
10-30
30-50
OVER 50

CONSISTENCY

SILTS AND CLAYS

VERY SOFT
SOFT
MEDIUM STIFF
STIFF
VERY STIFF
HARD

STRENGTH*

0-1/4
1/4-1/2
1/2-1
1-2
2-4
OVER 4

MOISTURE CONDITION

DRY
MOIST
WET

Dusty, dry to touch
Damp but no visible water
Visible freewater

LINE TYPES

————— Solid - Layer Break
----- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

Groundwater level during drilling
 Stabilized groundwater level

SAMPLER SYMBOLS

Modified California (3" O.D.) sampler
 California (2.5" O.D.) sampler
 S.P.T. - Split spoon sampler
 Shelby Tube
 Dames and Moore Piston
 Continuous Core
 Bag Samples
 Grab Samples
NR No Recovery

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
Expect Excellence



LOG OF BORING 1-B1

LATITUDE: 37.588639

LONGITUDE: -122.360325

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/13/2018
HOLE DEPTH: Approx. 61½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 12 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
10			LEAN CLAY WITH SAND (CL), dark yellowish brown, moist, medium plasticity, some fine sand, some organics [FILL]												
			CLAYEY SAND (SC), light yellowish brown, moist, medium plasticity, fine- to medium-grained sand [FILL]												
5			FAT CLAY WITH SAND (CH), grayish green, medium stiff, moist, high plasticity, trace organics and wood [FILL]												
5			Becomes soft			6	64	24	40				800*		TV
10						2									
0						<50 psi	87	33	54		55.3	67.2	732		UC
15			More gravels at 14'												
			CLAYEY SAND WITH GRAVEL (SC), dark yellowish brown to olive brown, medium dense, moist, medium plasticity, angular fine- to coarse-grained sand, clay			34				18					
-5															
20			Increasing fine gravel content, decreasing fines content.			20									
-10															
25															

OG - GEOTECHNICAL_SU+QU W/ ELEV 1095 ROLLINS RD.GPJ ENGEO INC.GDT 7/20/18



LOG OF BORING 1-B1

LATITUDE: 37.588639

LONGITUDE: -122.360325

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/13/2018
HOLE DEPTH: Approx. 61½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 12 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			SANDY CLAY WITH GRAVEL (CL), yellowish brown, stiff, moist, medium plasticity, fine to coarse sand, fine to medium gravel			53									
	-40														
			Decreasing gravel content			31									
	-45														
	55														
	-60		FAT CLAY (CH), grayish green, very stiff to hard, moist, high plasticity, trace organics and wood fragments												
			End boring at 61.5 feet below ground surface. Groundwater was not measured due to drilling method used.											3.5*	PP



LOG OF BORING 1-B2

LATITUDE: 37.588509

LONGITUDE: -122.359657

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 71.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 12 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			ASPHALT 3 inches thick												
			GRAVELLY CLAY (CL), dark grayish green, moist, medium plasticity [FILL]												
10			LEAN CLAY (CL), dark brownish gray, stiff, moist, medium plasticity, trace fine to medium angular sand [FILL]												
5			LEAN CLAY WITH SAND (CL), yellowish brown mottled with light grayish green, stiff, moist, medium plasticity, some fine to medium sand, trace fine gravel [FILL]			18	28	16	12	18.4			1.25*	PP	
5			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, dense, moist, fine to coarse gravel			13									
10						17				14	14.4	121.4	1214		UC
0															
15			SANDY LEAN CLAY (CL), yellowish brown to dark yellowish brown, stiff, moist, medium plasticity, fine to medium grained sand			18	39	20	19	80	23.8	103.4	5480		UU
-5															
20			Some fine to medium gravel			16	28	14	14		20.2	111.1	2104		UU
-10			Transitions to pale olive												
25															



LOG OF BORING 1-B2

LATITUDE: 37.588509

LONGITUDE: -122.359657

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 71.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 12 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			SANDY LEAN CLAY (CL), yellowish brown to dark yellowish brown, stiff, moist, medium plasticity, fine to medium grained sand			21	35	18	17		21.9	108.4	4680		UU
-15															
			CLAYEY SAND (SC), brown to dark brown, wet, fine to coarse sand, pockets of clayier material												
30															
			SANDY CLAY (CL), pale olive, soft to medium stiff, moist, medium plasticity, 15-20% fine to coarse sand			12								4.5*	PP
-20			LEAN CLAY (CL), brown, very stiff to hard, slightly moist to moist, medium plasticity, veins of black												
35			CLAYEY SAND WITH GRAVEL (SC), yellowish red to dark yellowish brown, stiff to very soft, moist, medium plasticity, fine to coarse sand, fine to medium gravel, active weathering of gravels			24				43				2.25*	PP
-25															
			Becoming more plastic, decreasing sand content												
40			CLAY (CL), olive brown to yellowish brown, very stiff to hard, moist, medium plasticity, trace fine to coarse sand and fine gravel, veins of black and white weathering			13								3.0*	PP
-30															
			Transitions to pale olive mottled with yellowish red			18	47	22	25		29.7			1.5*	PP
45															
-35															
50															



LOG OF BORING 1-B2

LATITUDE: 37.588509

LONGITUDE: -122.359657

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 71.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 12 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			SANDY SILT (ML), olive brown, medium dense, saturated, low plasticity, rapid dilatancy, fine sand, trace coarse sand			14									
	-40		Sands and gravels in cuttings												
			CLAYEY SAND WITH GRAVEL (SC), olive brown, dense, wet, fine to coarse sand, fine to coarse gravel			34									
	-45														
			FAT CLAY (CH), grayish green, very stiff, moist, high plasticity, trace organics, wood fragments, trace fine to medium sand			17								2.75*	PP
	-50														
			Trace coarse sand			38								1.5*	PP
	-55														
			Increasing fine- to coarse-grained sand content			46									
	-70														
			End boring at 71.5 feet below ground surface. Groundwater was not measured due to drilling method used.											3.0*	PP



LOG OF BORING 1-B3

LATITUDE: 37.588131

LONGITUDE: -122.359511

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			ASPHALT 3 inches thick												
			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, slightly moist, fine to medium gravel. [FILL]												
			CLAYEY SAND (SC), yellowish brown, slightly moist, 10-20% fines content. [FILL]												
10			LEAN CLAY WITH SAND AND GRAVEL (CL), dark brown to dark yellowish brown, medium stiff, moist, medium plasticity, fine to coarse sand, fine grained gravel. [FILL]			21								0.75*	PP
5															
5			SANDY CLAY WITH GRAVEL (CL), yellowish brown, soft, very moist, medium plasticity, 20-30% fine to coarse sand, 10-20% fine to coarse gravel, max size 2". [FILL]			6									
10			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, loose, very moist, sand grains are angular to subangular, some fine gravel.			9	27	18	9	24					
0															
15						19									
-5															
20			LEAN CLAY WITH SAND (CL), dark reddish brown mottled with yellowish brown, very stiff, moist, medium plasticity, medium coarse subrounded sand.			28								3.5*	PP
-10															
25															



LOG OF BORING 1-B3

LATITUDE: 37.588131

LONGITUDE: -122.359511

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, medium dense, moist, clay is medium plasticity, fine gravel.			46									
-15															
30			LEAN CLAY WITH SAND (CL), pale olive to pale yellow, very stiff, moist, medium plasticity, fine to coarse sand, trace fine gravel, active weathering of gravels.			9								2.0*	PP
-20			SANDY CLAY (CL), yellowish brown, stiff to very stiff, moist, low to medium plasticity, 15-25% fine sand.			27								3.0*	PP
														1.5*	PP
35															
-25															
40			CLAYEY SAND WITH GRAVEL (SC), dark yellowish brown, very stiff, wet, medium plasticity, fine to coarse sand, 15-20% fines, 5% fine gravel			26								2.5*	PP
			SANDY LEAN CLAY (CL), dark yellowish brown, very stiff, moist, medium plasticity, 25-30% fine sand												
-30															
45			CLAYEY GRAVEL WITH SAND (GC), yellowish brown, dense, wet, medium plasticity, fine to coarse gravel, max size 2", fine to coarse sand			47									
-35															
50															



LOG OF BORING 1-B3

LATITUDE: 37.588131

LONGITUDE: -122.359511

Geotechnical Exploration
1095 Rollins Road
Burlingame, CA
14140.000.000

DATE DRILLED: 6/12/2018
HOLE DEPTH: 61.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (WGS84): Approx. 13 ft.

LOGGED / REVIEWED BY: S. Waganaar / TB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

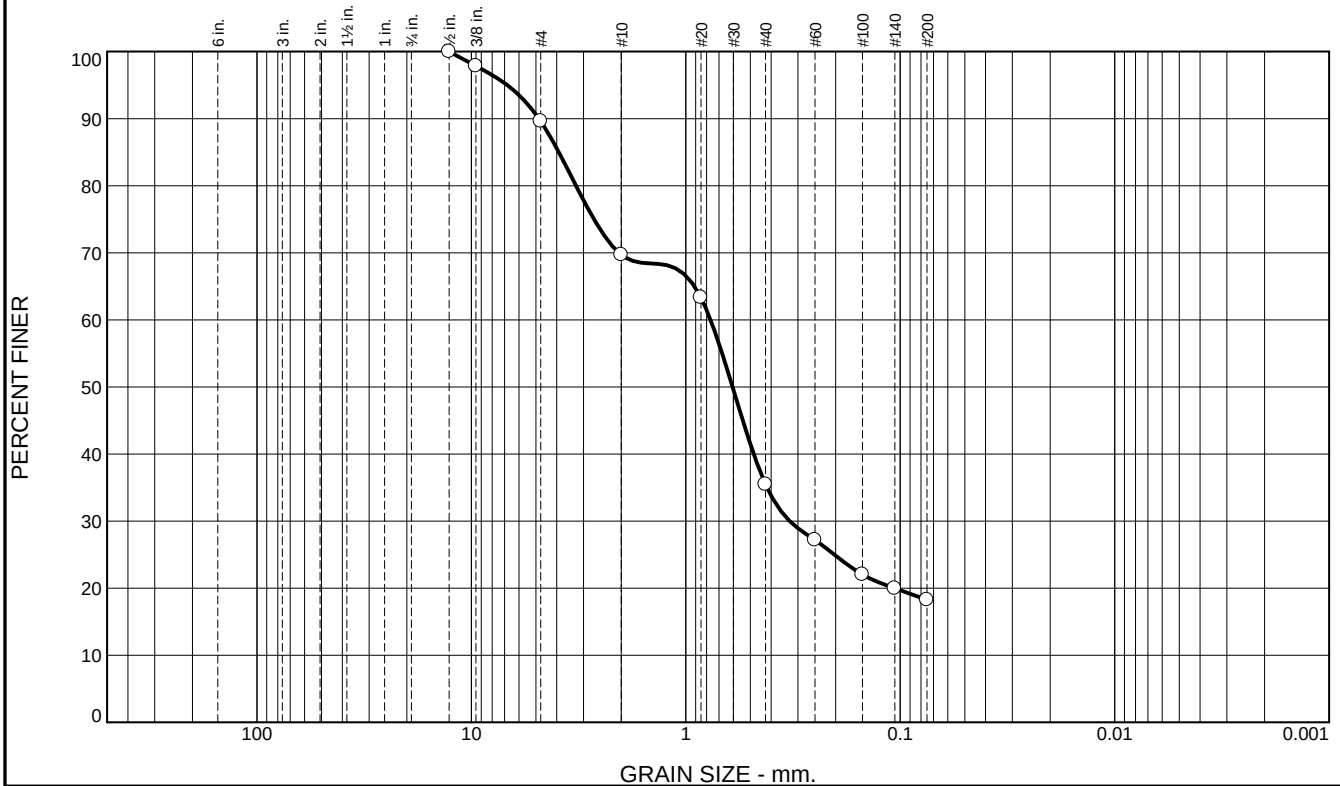
Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
							Liquid Limit	Plastic Limit	Plasticity Index						
			CLAYEY SAND WITH GRAVEL (SC), yellowish brown, dense, wet, medium plasticity, fine to coarse sand, fine gravel			35									
	-40														
			POORLY GRADED GRAVEL WITH CLAY (GP), yellowish brown to brown, dense, wet, subrounded, 5-10% fines												
55															
			CLAYEY SAND (SC), yellowish brown, dense, wet, medium plasticity, subangular, angular, fine to coarse gravel			45									
	-45														
			FAT CLAY (CH), grayish green, very stiff, moist, high plasticity, fine sand, trace organics			28									
60															
			End boring at 61.5 feet below ground surface. Groundwater was not measured due to drilling method used.											3.0*	PP



APPENDIX B

LABORATORY TEST DATA

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	10.4	19.9	34.2	17.2	18.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	97.9		
#4	89.6		
#10	69.7		
#20	63.3		
#40	35.5		
#60	27.2		
#100	22.0		
#140	20.0		
#200	18.3		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 4.8440 D₈₅= 3.9150 D₆₀= 0.7666
 D₅₀= 0.6053 D₃₀= 0.3280 D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D6913, Method B

Sample Number: 1-B1 @ 16

Depth: 16.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

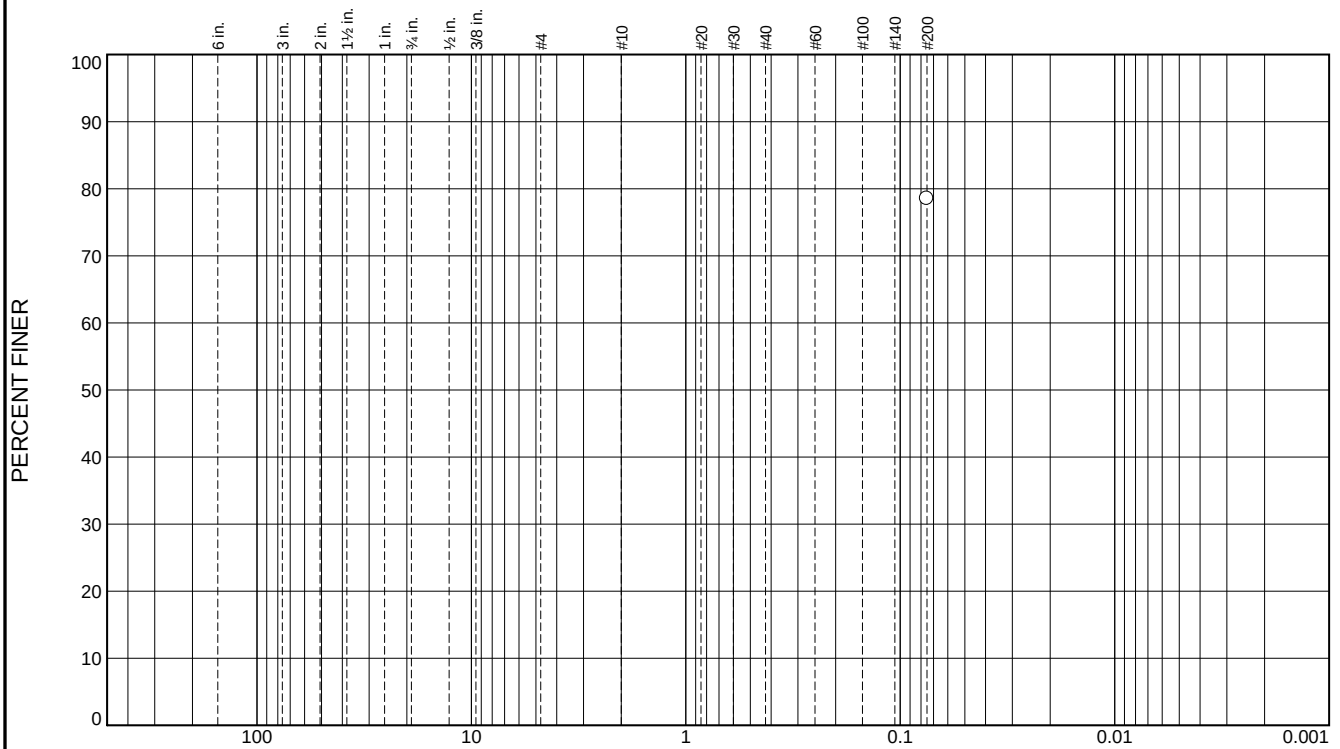
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						78.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	78.5		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D1140, Method B
 Dry Sample Weight = 88.59; Soak Time = 4 hrs

Sample Number: 1-B1 @ 31

Depth: 31 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

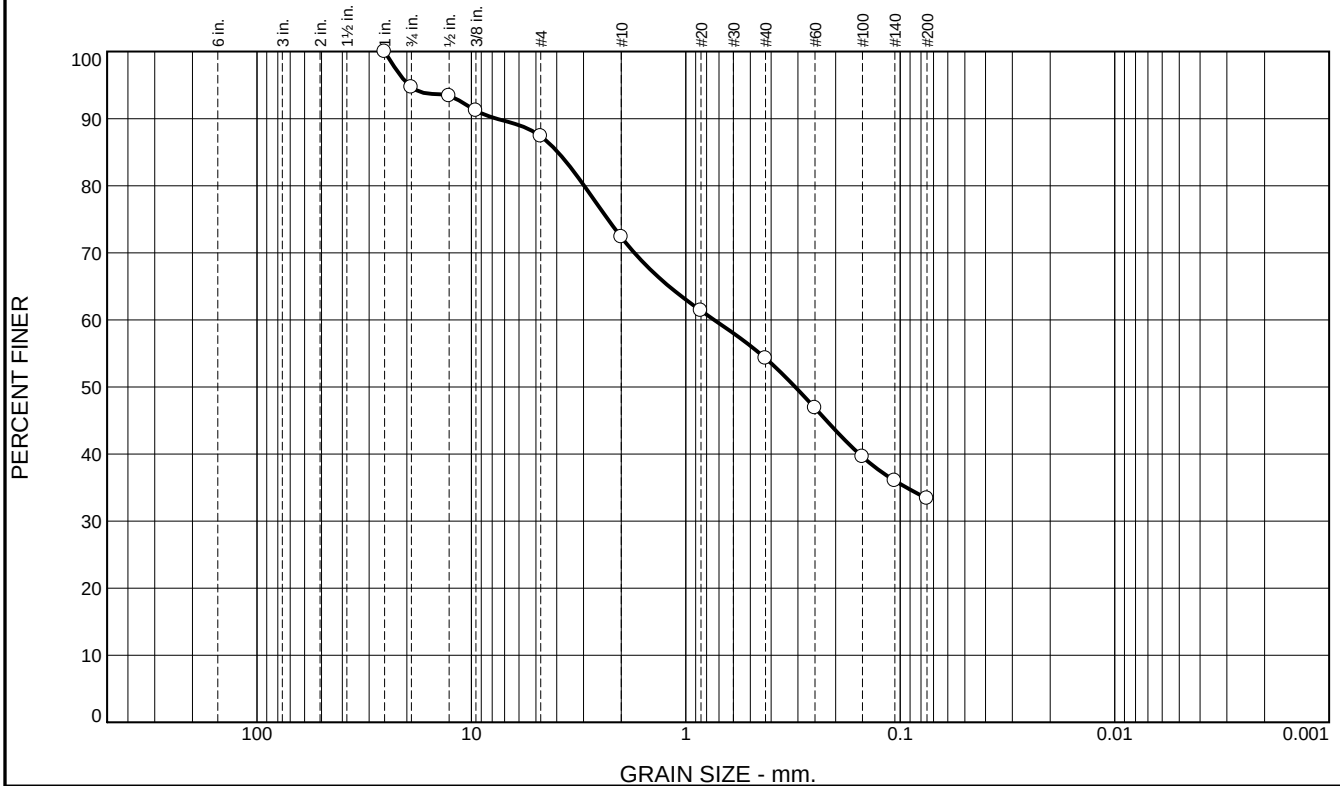
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.3	7.3	15.1	18.0	20.9	33.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	94.7		
1/2	93.4		
3/8	91.2		
#4	87.4		
#10	72.3		
#20	61.4		
#40	54.3		
#60	46.9		
#100	39.6		
#140	36.0		
#200	33.4		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 7.6381 D₈₅= 3.9508 D₆₀= 0.7375
 D₅₀= 0.3091 D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D6913, Method B

Sample Number: 1-B1 @ 36

Depth: 36.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

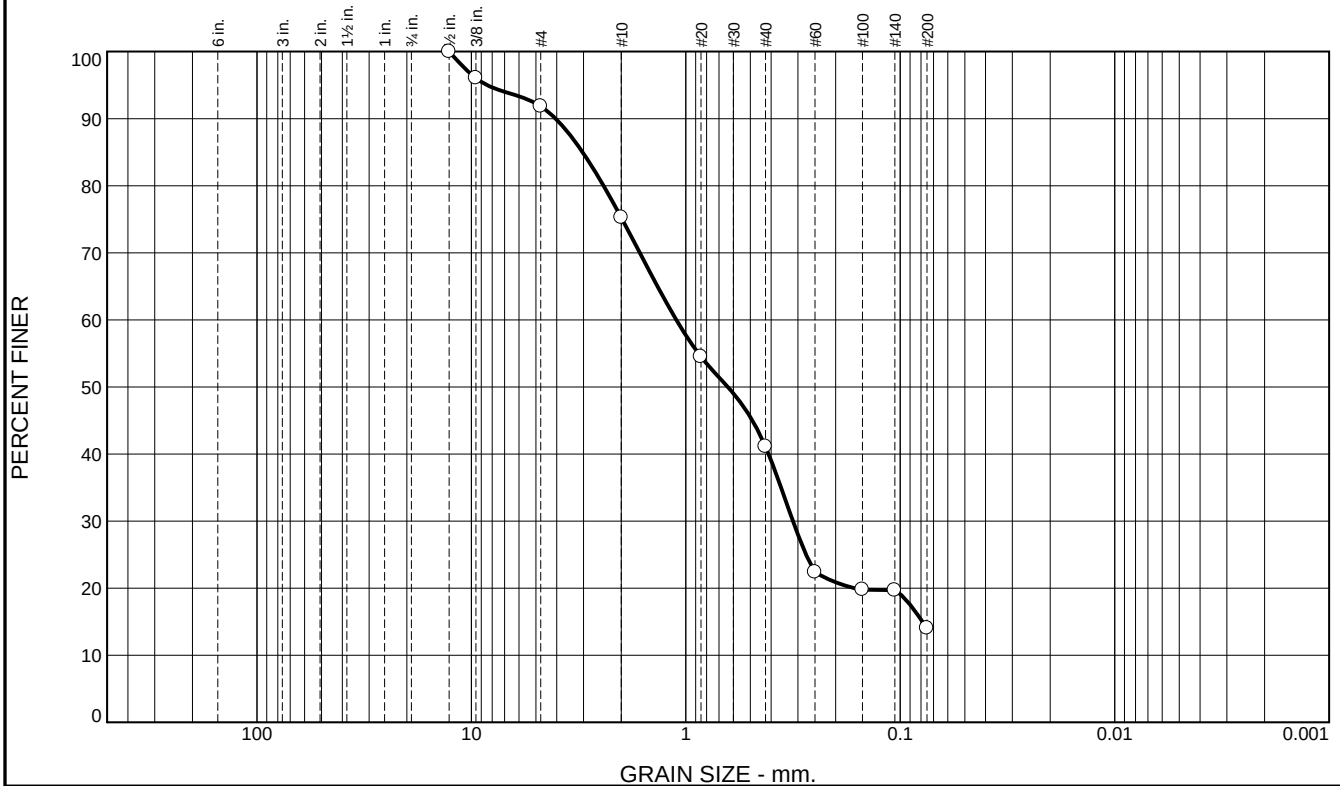
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.2	16.5	34.2	27.1	14.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	96.0		
#4	91.8		
#10	75.3		
#20	54.5		
#40	41.1		
#60	22.4		
#100	19.8		
#140	19.7		
#200	14.0		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 4.0500 D₈₅= 3.0289 D₆₀= 1.1076
 D₅₀= 0.6370 D₃₀= 0.3168 D₁₅= 0.0787
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D6913, Method B

Sample Number: 1-B2 @ 11

Depth: 11.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

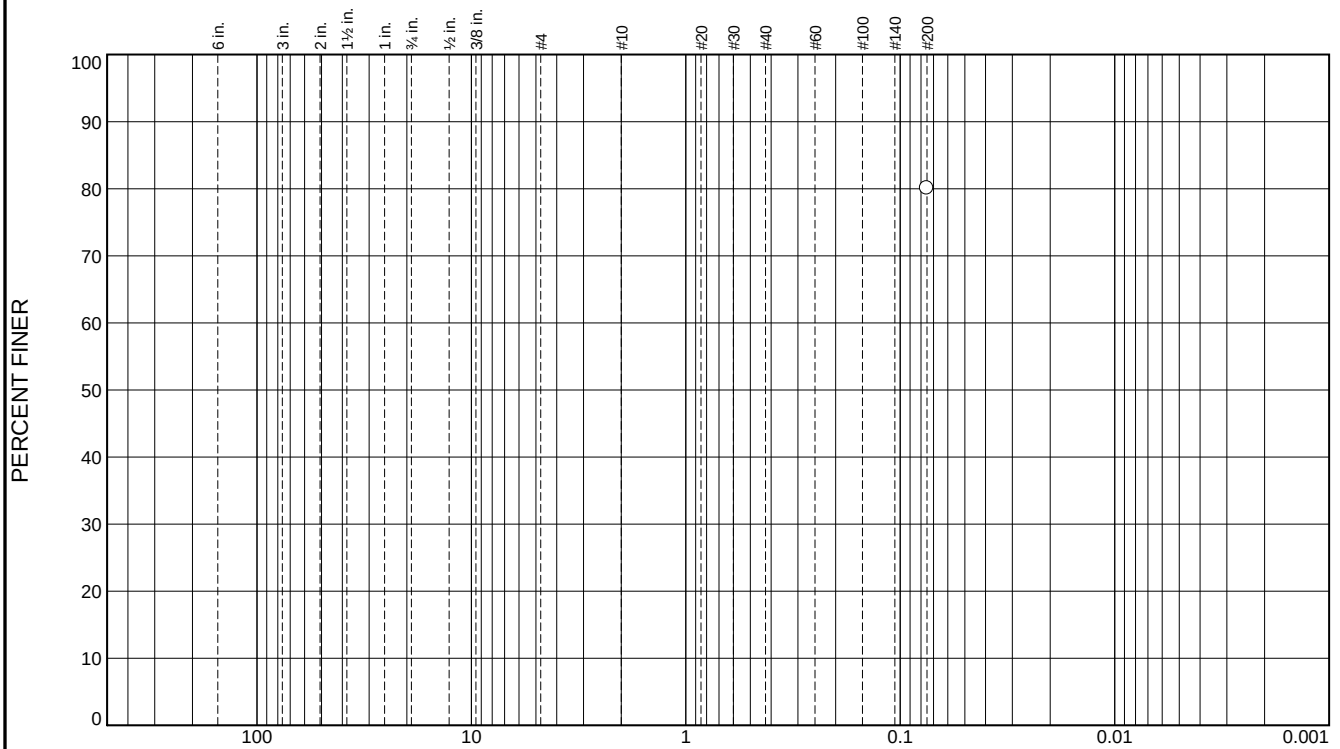
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						80.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	80.1		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= 20 LL= 39 PI= 19

Coefficients
 D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 GS: ASTM D1140, Method B
 Dry Sample Weight = 121.78; Soak Time = 4 hrs
 PI: ASTM D4318, Wet method

Sample Number: 1-B2 @ 16

Depth: 16.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

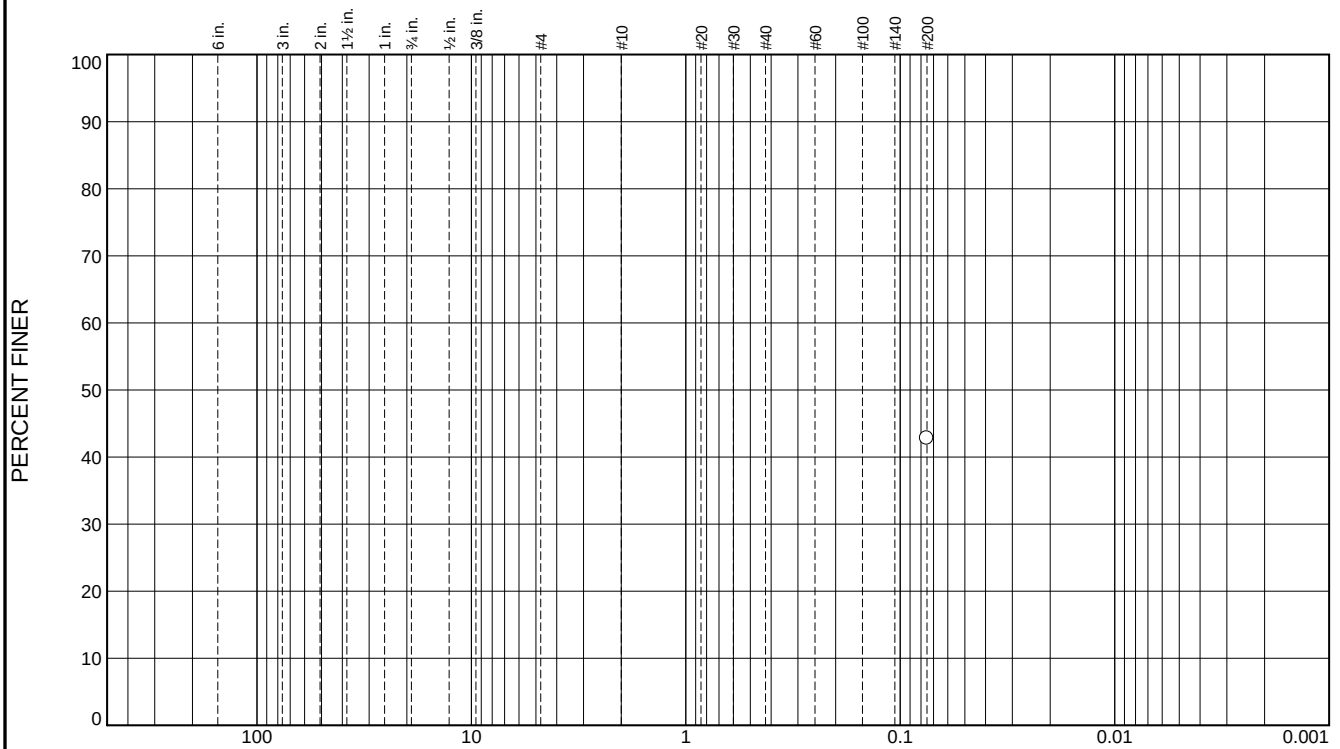
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						42.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	42.8		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D1140, Method B
 Dry Sample Weight = 796.3; Soak Time = 4 hrs

Sample Number: 1-B2 @ 36

Depth: 36.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

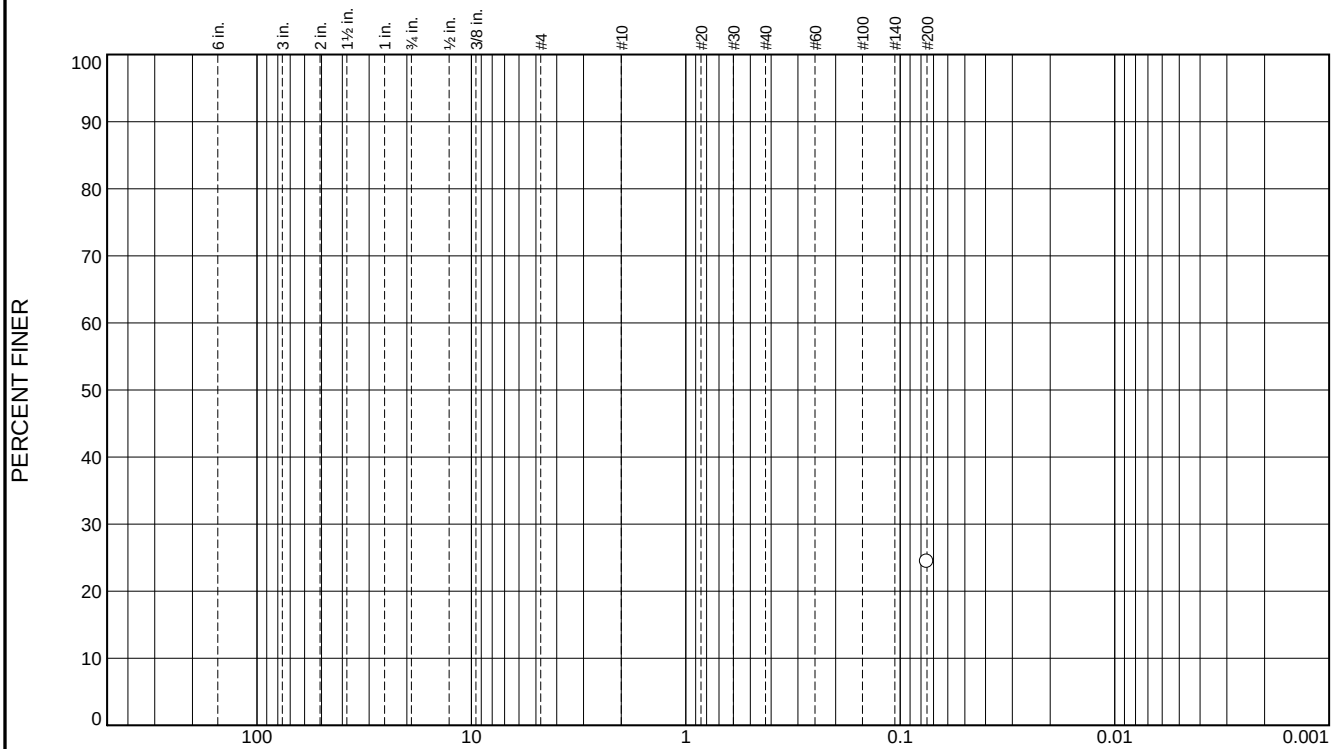
Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Quasem

Checked By: M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						24.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	24.4		

* (no specification provided)

Material Description
See exploration logs

Atterberg Limits
 PL= 18 LL= 27 PI= 9

Coefficients
 D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 GS: ASTM D1140, Method B
 Dry Sample Weight = 120.11; Soak Time = 4 hrs
 PI: ASTM D4318, Wet method

Sample Number: 1-B3 @ 10

Depth: 10.0 feet

Date: 7/17/2018



Client: Hanover R.S. Limited Partnership

Project: 1095 Rollins Rd., Burlingame

Project No: 14140.000.000

Tested By: M. Bromfield

Checked By: M. Quasem

MOISTURE CONTENT DETERMINATION

ASTM D2216

BORING/SAMPLE ID	1-B2	1-B2						
DEPTH (ft)	5.0	46.0						
Method A or B	B	B						
% <i>MOISTURE</i>	18.4	29.7						

PROJECT NAME: 1095 Rollins Rd, Burlingame

PROJECT NUMBER: 14140.000.000

CLIENT: Hanover R.S. Limited Partnership

PHASE NUMBER: 002

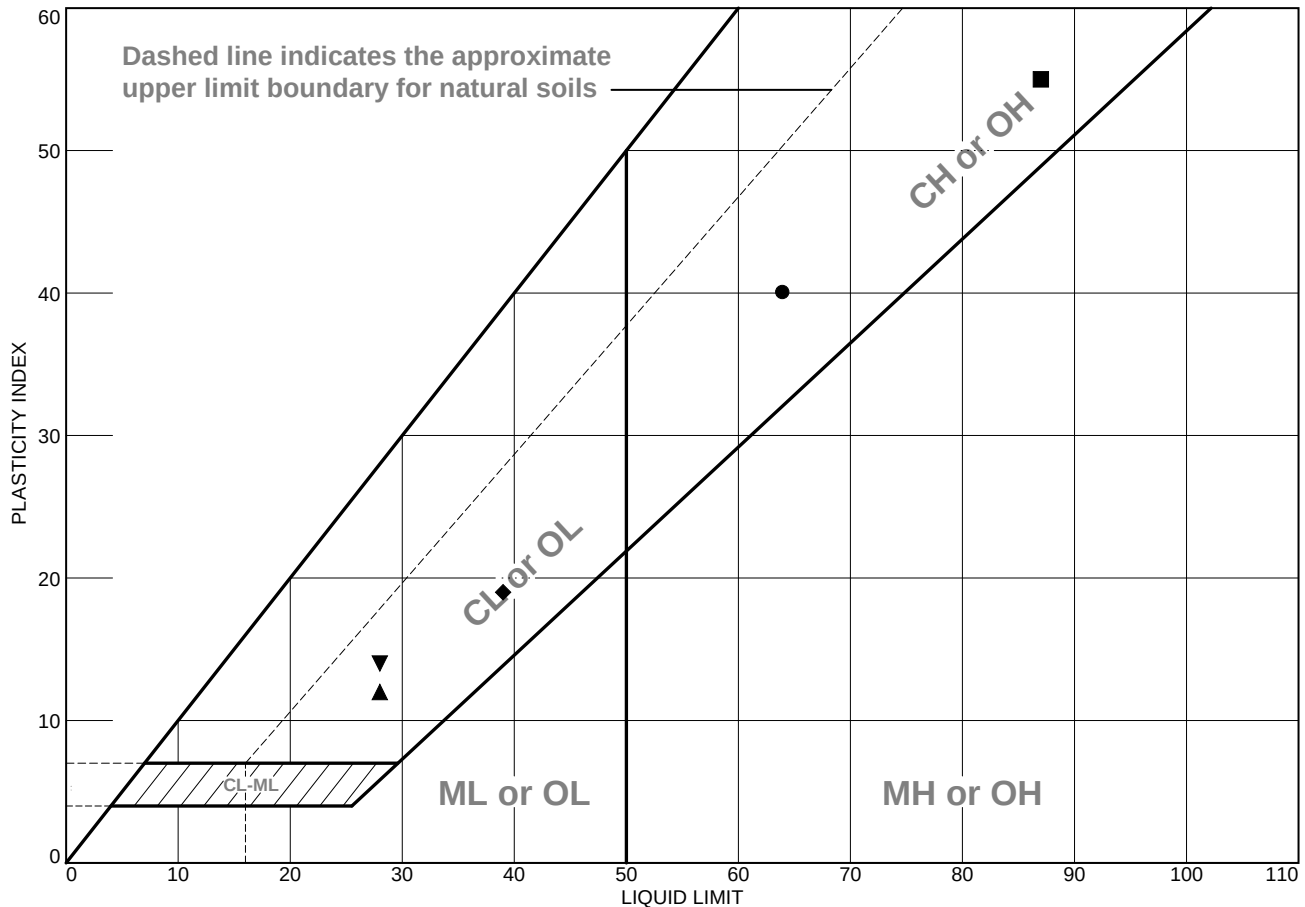
DATE: 07/10/18



Tested by: M. Bromfield

Reviewed by: M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	64	24	40			
■	See exploration logs	87	32	55			
▲	See exploration logs	28	16	12			
◆	See exploration logs	39	20	19		80.1	
▼	See exploration logs	28	14	14			

Project No. 14140.000.000 **Client:** Hanover R.S. Limited Partnership

Project: 1095 Rollins Rd., Burlingame

● **Depth:** 6.0 feet **Sample Number:** 1-B1 @ 6
 ■ **Depth:** 11.0 feet **Sample Number:** 1-B1 @ 11
 ▲ **Depth:** 5.0 feet **Sample Number:** 1-B2 @ 5
 ◆ **Depth:** 16.0 feet **Sample Number:** 1-B2 @ 16
 ▼ **Depth:** 21.0 feet **Sample Number:** 1-B2 @ 21

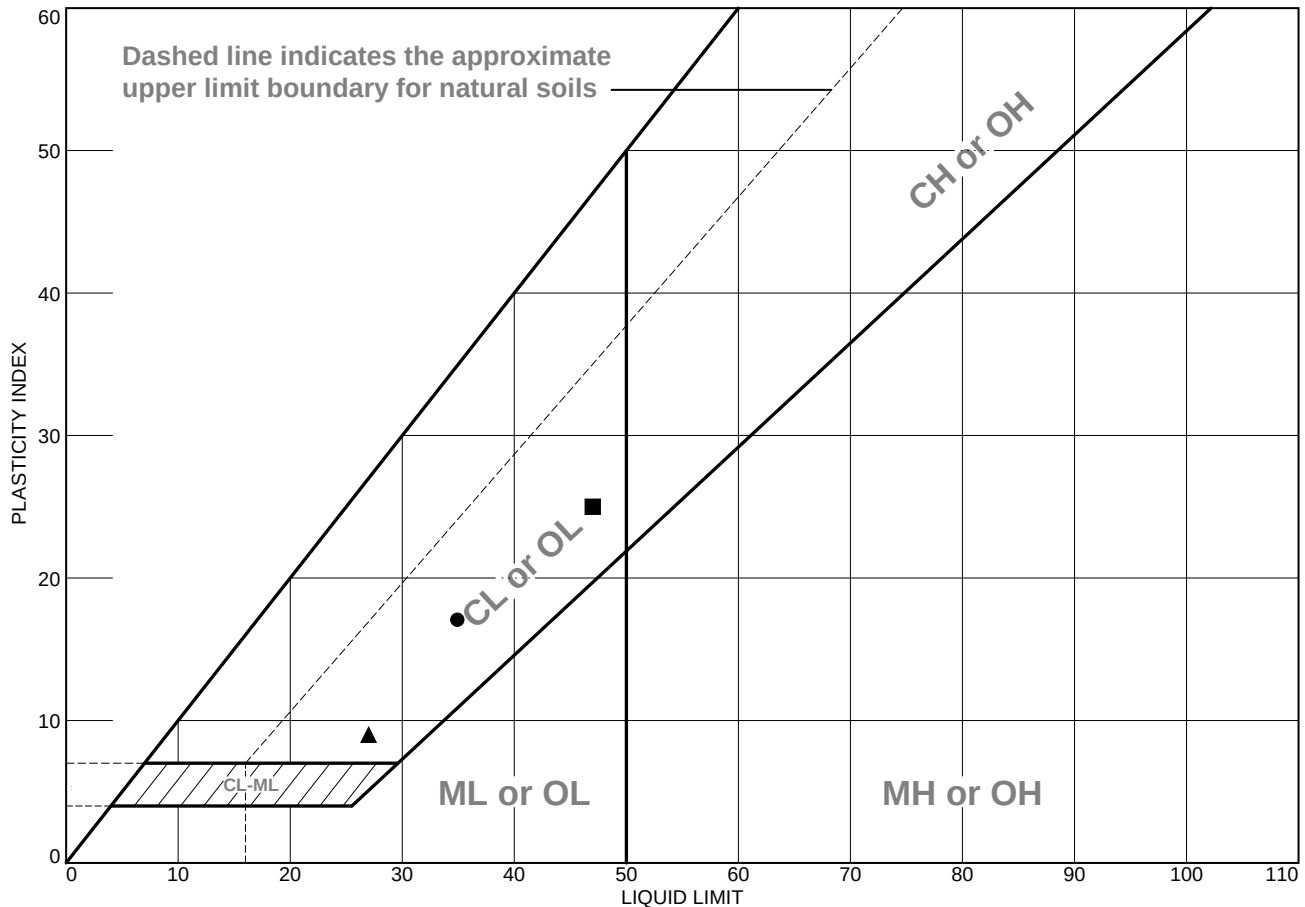
ENGEO
INCORPORATED

Remarks:

● ASTM D4318, Wet method
 ■ ASTM D4318, Wet method
 ▲ ASTM D4318, Wet method
 ◆ PI: ASTM D4318, Wet method
 GS: ASTM D1140, Method B
 ▼ ASTM D4318, Wet method

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	35	18	17			
■	See exploration logs	47	22	25			
▲	See exploration logs	27	18	9		24.4	

Project No. 14140.000.000 **Client:** Hanover R.S. Limited Partnership

Project: 1095 Rollins Rd., Burlingame

● **Depth:** 26.0 feet **Sample Number:** 1-B2 @ 26
 ■ **Depth:** 46.0 feet **Sample Number:** 1-B2 @ 46
 ▲ **Depth:** 10.0 feet **Sample Number:** 1-B3 @ 10

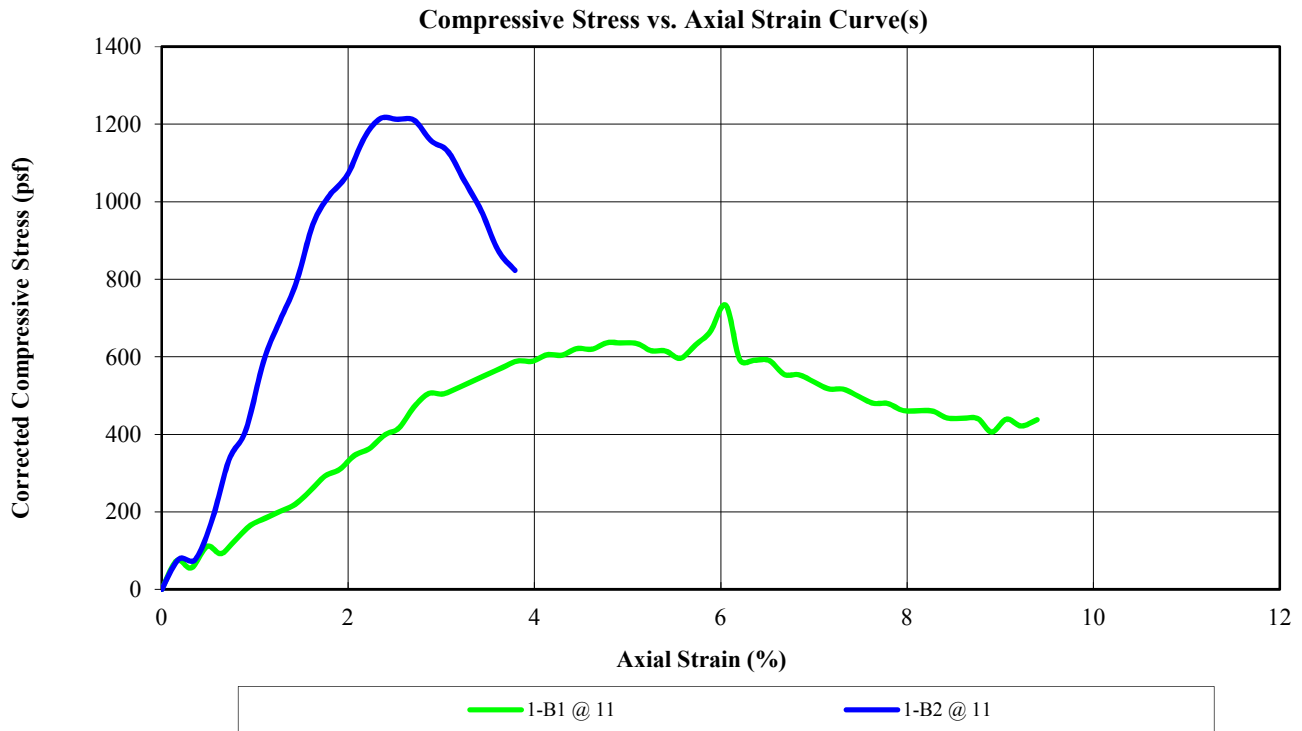


Remarks:

● ASTM D4318, Wet method
 ■ ASTM D4318, Wet method
 ▲ PI: ASTM D4318, Wet method
 GS: ASTM D1140, Method B

Tested By: ○ M. Bromfield □ M. Quasem △ M. Bromfield **Checked By:** M. Quasem

UNCONFINED COMPRESSION TEST REPORT (ASTM D2166)



BEFORE TEST	SPECIMEN	
	1-B1 @ 11	1-B2 @ 11
Moisture Content (%)	55.3	14.5
Dry Density (pcf)	67.2	121.4
Saturation (%)	100.0	100.0
Void Ratio	1.46	0.36
Diameter (in)	2.853	2.415
Height (in)	6.46	5.69
Height-To-Diameter Ratio	2.26	2.36
TEST DATA		
Unconfined Compressive Strength (psf)	733	1215
Undrained Shear Strength (psf)	366	607
Strain Rate (in./min.)	0.05	0.05
Specific Gravity	2.650	2.650
Strain at Failure (%)	6.05	2.35
Liquid Limit	-	-
Plastic Limit	-	-
Test Remarks		
SPECIMEN	DESCRIPTION	
1-B1 @ 11	See exploration logs	
1-B2 @ 11	See exploration logs	

PROJECT NAME: 1095 Rollins Rd, Burlingame

Test Date: 07/12/18

PROJECT NO: 14140.000.000

Tested By: M. Bromfield

CLIENT: Hanover R.S. Limited Partnership

Reviewed By: M. Quasem

LOCATION: Burlingame, CA

PHASE NO: 002



Unconsolidated Undrained Triaxial Test (ASTM D2850)

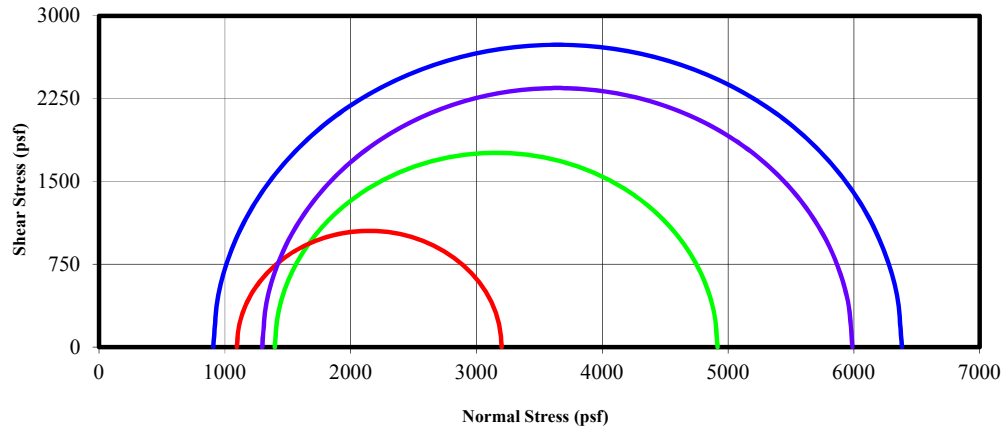
Date: 07/13/18

Checked By: G. Criste

Date: 07/13/18

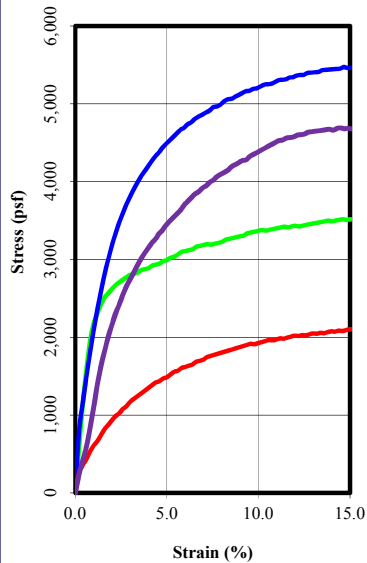
Tested By: M. Quasem

Mohr Circles



— 1-B1@31
 — 1-B2@16
 — 1-B2@21
 — 1-B2@26

Stress-Strain Curve



Specimen				
Before Test	1-B2@31	1-B2@16	1-B2@21	1-B2@26
Water Content (%)	29.05	23.76	20.19	21.87
Dry Density (pcf)	96.41	103.21	111.06	108.44
Saturation (%)	100.00	100.00	100.00	100.00
Void Ratio	0.72	0.60	0.49	0.53
Diameter (in)	2.410	2.429	2.410	2.423
Height (in)	5.031	5.078	5.040	5.070
Liquid Limit	-	-	-	-
Plastic Limit	-	-	-	-
Specific Gravity	2.650	2.650	2.650	2.650
Height-to-Diameter Ratio	2.088	2.091	2.091	2.092
After Test	1-B2@31	1-B2@16	1-B2@21	1-B2@26
Water Content (%)	29.05	23.76	20.19	21.87
Saturation (%)	100.00	100.00	100.00	100.00
Strain Rate (in/min)	0.05	0.05	0.05	0.05
Peak Deviator Stress (psf)	3518.9	5474.7	2105.3	4692.1
Axial Strain @ Failure (%)	15.008	14.670	14.981	14.495
Cell Pressure				
Cell (psf)	1396.8	907.2	1094.4	1296.0
Back (psf)	n/a	n/a	n/a	n/a
Principle Stresses at Failure				
σ_1 (psf)	4915.7	6381.9	3199.7	5988.1
σ_3 (psf)	1396.8	907.2	1094.4	1296.0

Mohr-Coulomb Parameters with a Non-zero Friction Angle ($\phi \neq 0$)			Cohesion at Failure with a Zero Friction Angle ($\phi=0$)			
Cohesion, c (psf)	0.0		1759.5	2737.4	1052.6	2346.0
Friction Angle ϕ	0.00		n/a	n/a	n/a	n/a
Project Information						
Project Name:	1095 Rollins Rd, Burlingame					
Project Number:	14140.000.000		Job Number:		14140.000.000	
Location:	Burlingame, CA		Boring Number:		Multiple	
Client:	Hanover R.S. Limited Partnership		Sample Number:		Multiple	
Description:	See exploration logs					

Unconsolidated Undrained Triaxial Test (ASTM D2850)

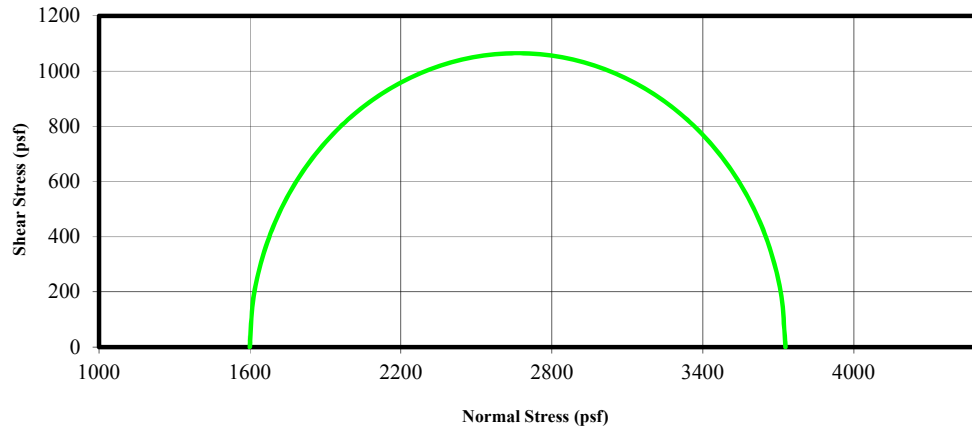
Date: 07/16/18

Checked By: G. Criste

Date: 07/16/18

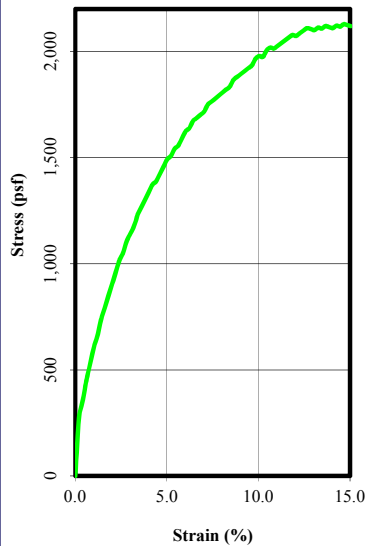
Tested By: M. Quasem

Mohr Circles



1-B3@36

Stress-Strain Curve



Specimen				
Before Test		1-B3@36		
Water Content (%)	19.66			
Dry Density (pcf)	111.48			
Saturation (%)	100.00			
Void Ratio	0.48			
Diameter (in)	2.420			
Height (in)	5.010			
Liquid Limit	-			
Plastic Limit	-			
Specific Gravity	2.650			
Height-to-Diameter Ratio	2.070			
After Test		1-B3@36		
Water Content (%)	19.66			
Saturation (%)	100.00			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	2129.1			
Axial Strain @ Failure (%)	14.669			
Cell Pressure				
Cell (psf)	1598.4			
Back (psf)	n/a			
Principle Stresses at Failure				
σ_1 (psf)	3727.5			
σ_3 (psf)	1598.4			

Mohr-Coulomb Parameters with a Non-zero Friction Angle ($\phi \neq 0$)			Cohesion at Failure with a Zero Friction Angle ($\phi=0$)			
Cohesion, c (psf)	0.0		1064.6			
Friction Angle ϕ	0.00		n/a			
Project Information						
Project Name:	1095 Rollins Rd, Burlingame					
Project Number:	14140.000.000		Job Number:		14140.000.000	
Location:	Burlingame, CA		Boring Number:		1-B3	
Client:	Hanover R.S. Limited Partnership		Sample Number:		1-B3@36	
Description:	See exploration logs					



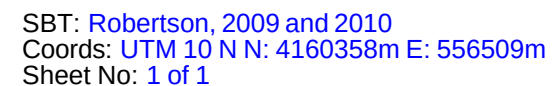
APPENDIX C

CPT DATA



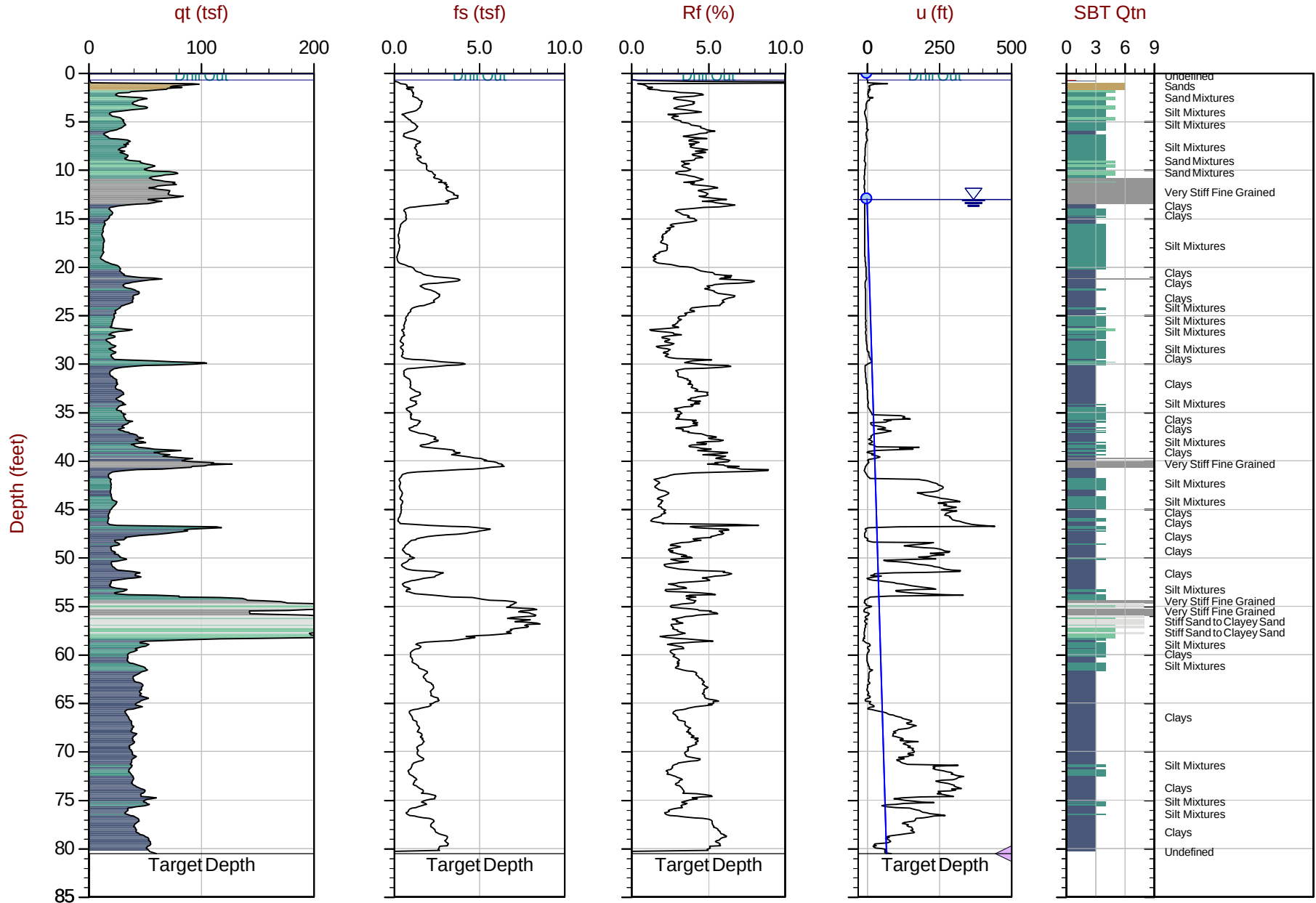
Site: 1095 Rollins Rd, Burlingame

Cone: 446:T1500F15U500



● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◀ Dissipation, Ueq achieved
 ◀ Dissipation, Ueq not achieved
 — Hydrostatic Line

The reported coordinates were acquired from Google Earth imagery and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 24.550 m / 80.54 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 18-56095_CP02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 10 N N: 4160351m E: 556534m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

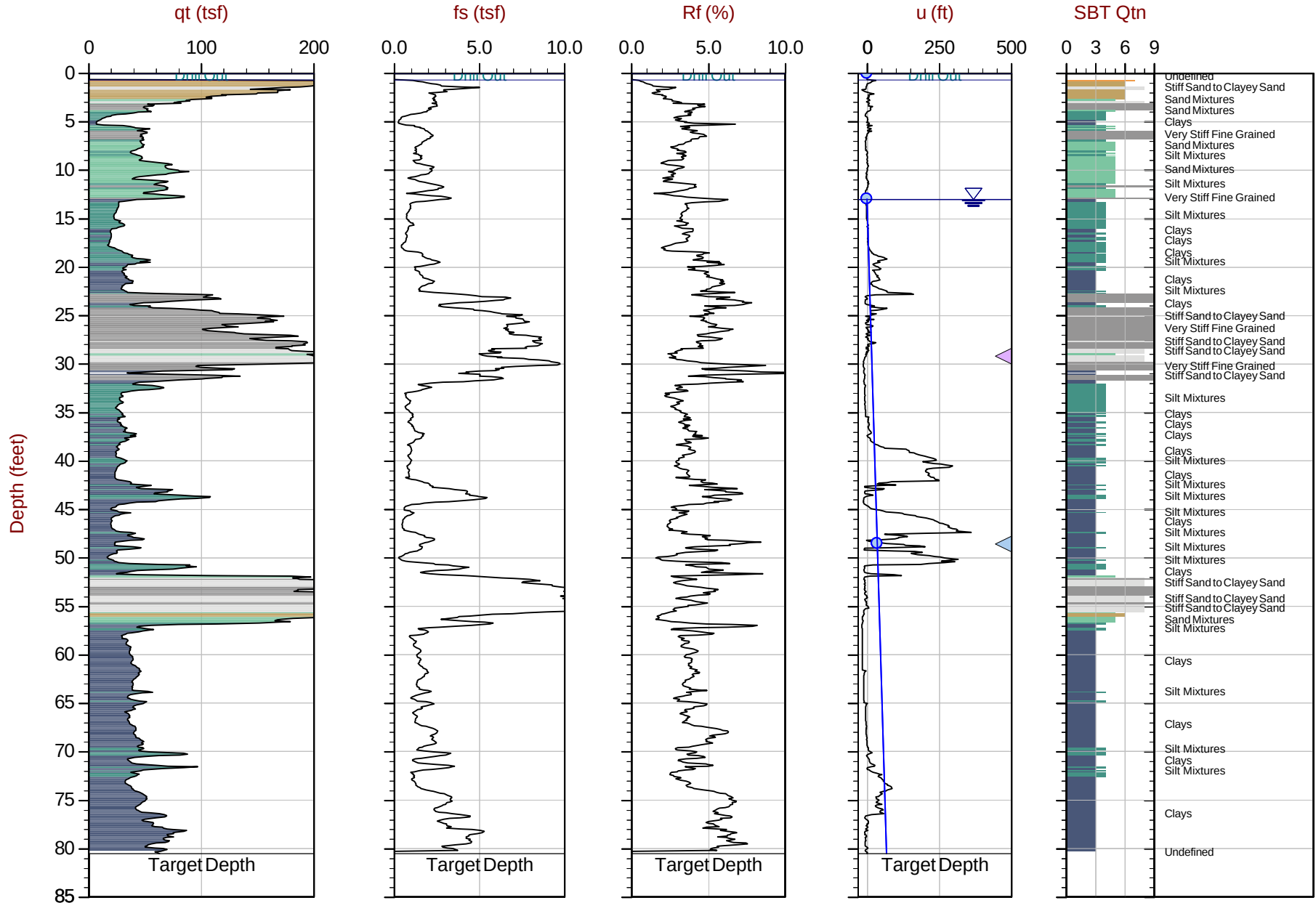
● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from Google Earth imagery and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 24.550 m / 80.54 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 18-56095_CP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10 N N: 4160323m E: 556526m
Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)
 ● Assumed Ueq
 ◀ Dissipation, Ueq achieved
 ◀ Dissipation, Ueq not achieved
 — Hydrostatic Line

The reported coordinates were acquired from Google Earth imagery and are only approximate locations. The coordinates should not be used for design purposes.



APPENDIX D

LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS REPORT

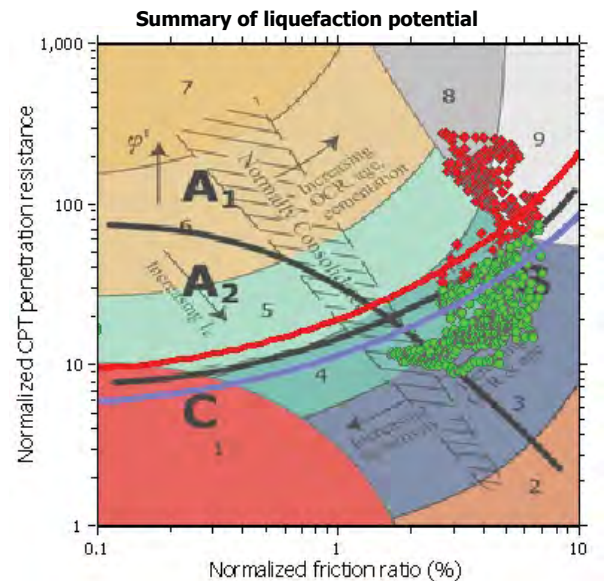
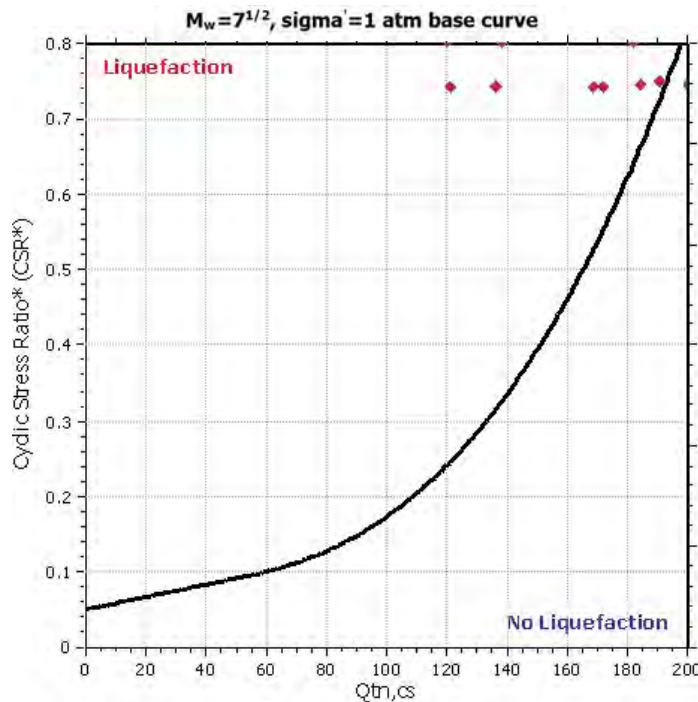
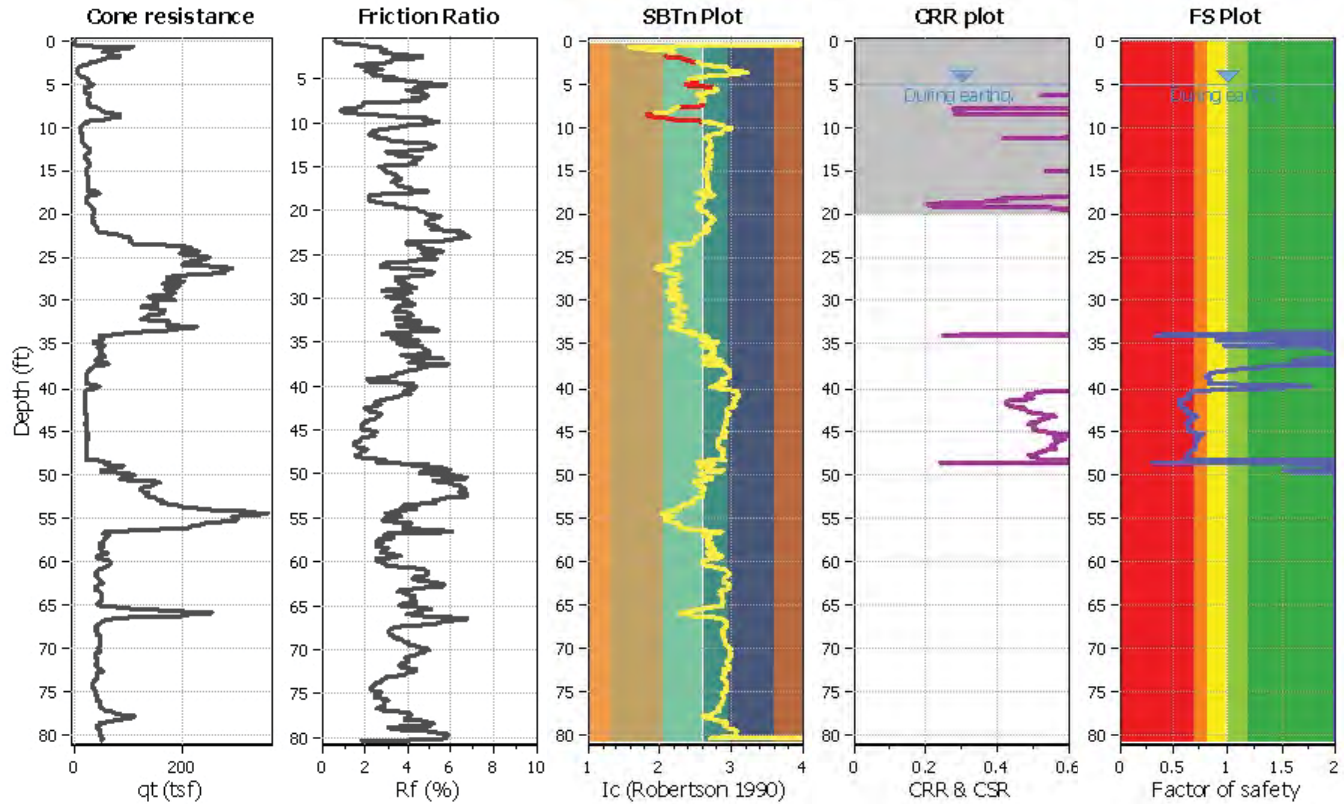
Project title : 1095 Rollins Road, 14140.000.000

Location : Burlingame, California

CPT file : 1-CPT1

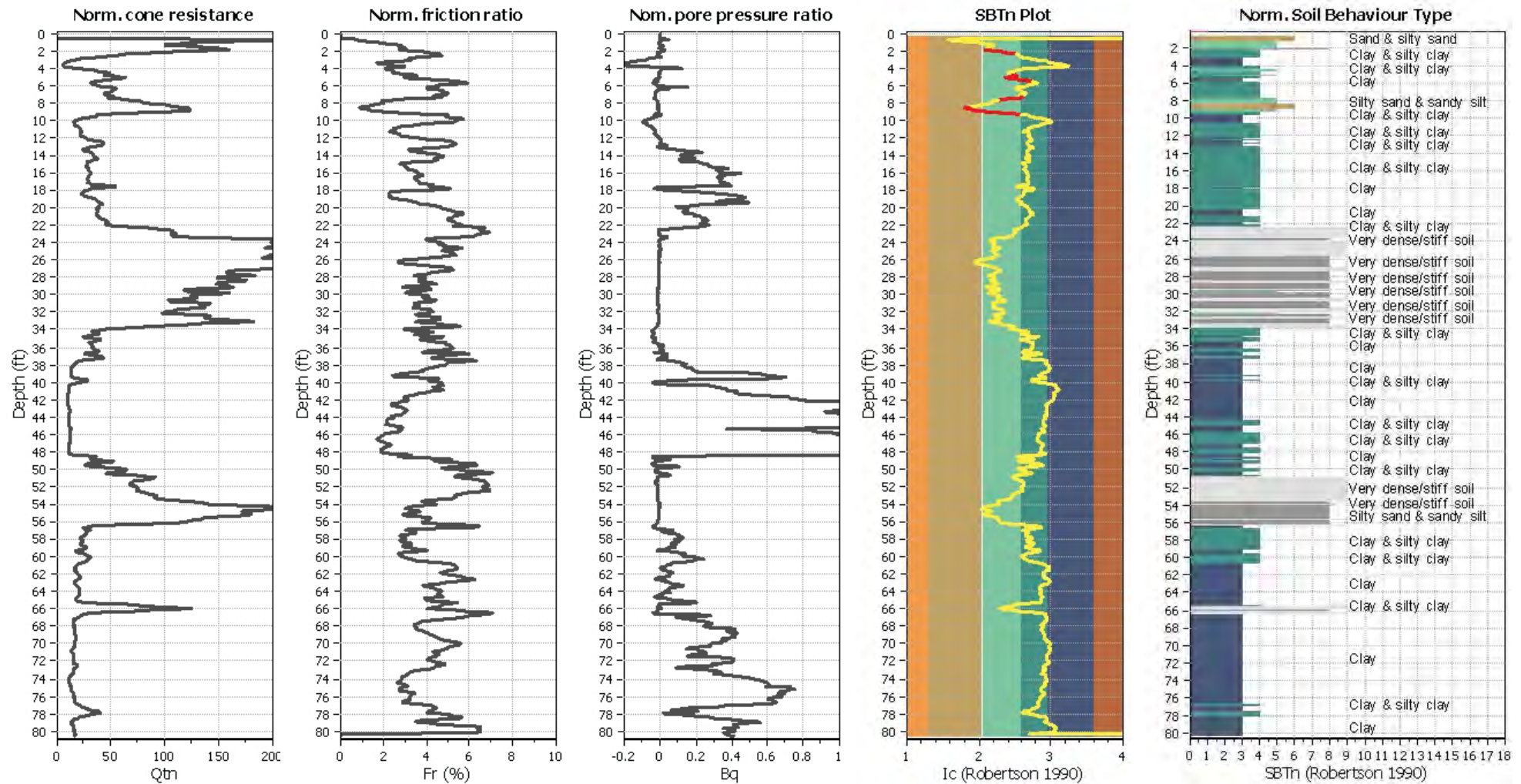
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Excavation:	Yes	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Excavation depth:	20.00 ft	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	2.00 tsf	Limit depth applied:	Yes
Earthquake magnitude M_w :	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_0 applied:	No	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz

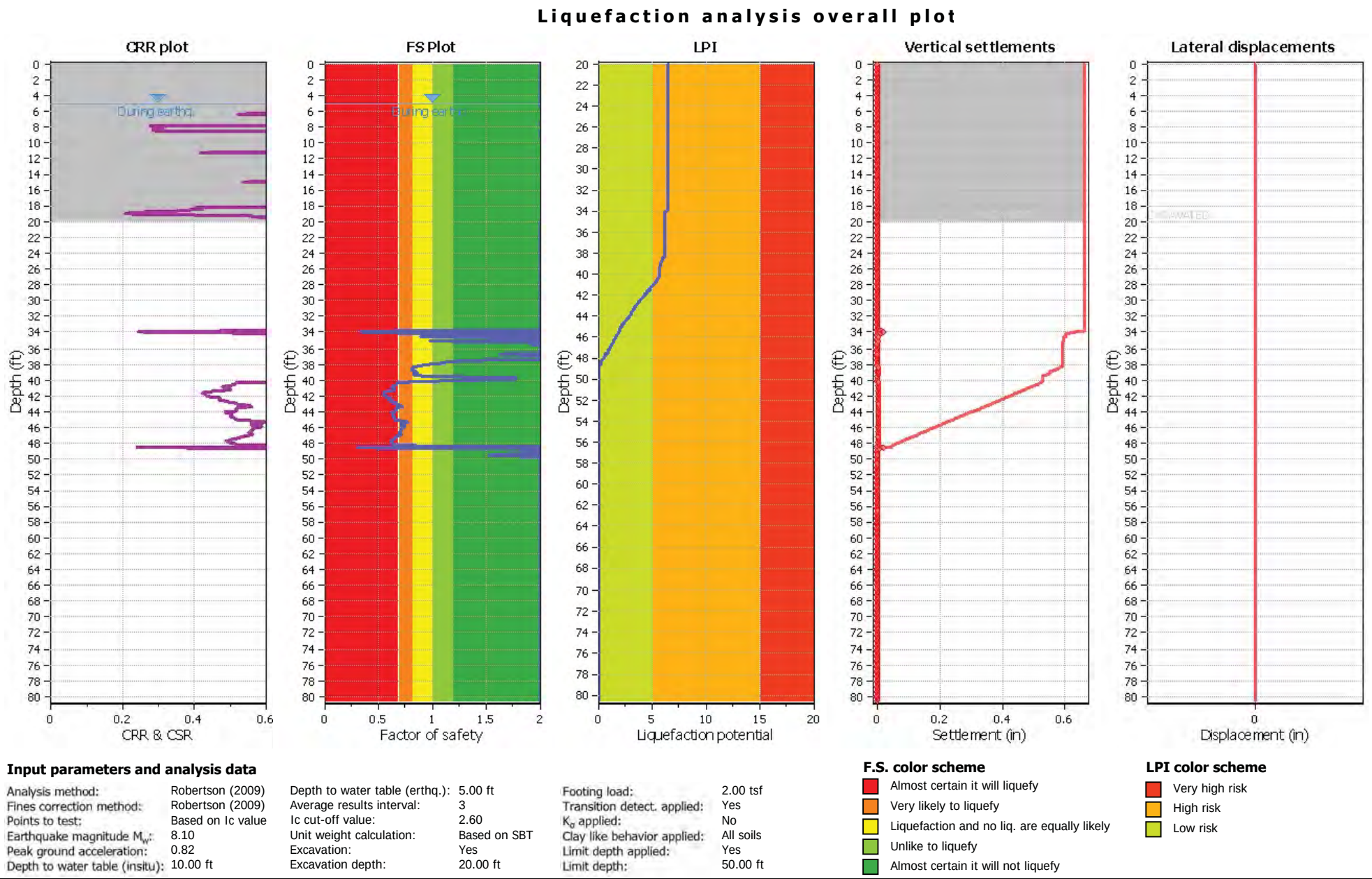


Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.82	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Excavation depth:	20.00 ft	Limit depth:	50.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained



LIQUEFACTION ANALYSIS REPORT

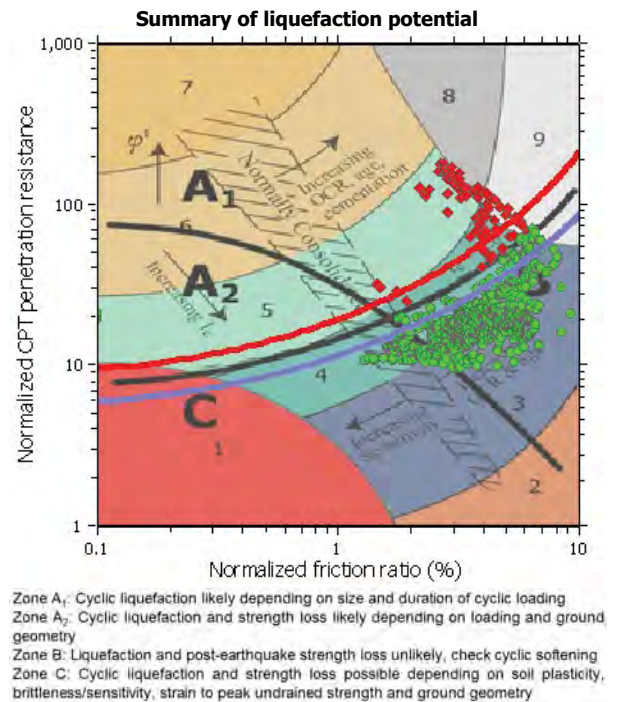
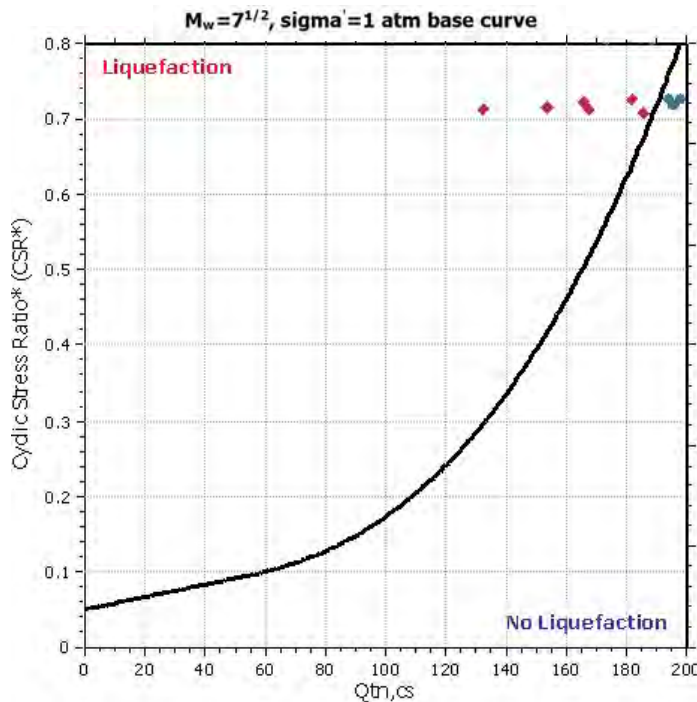
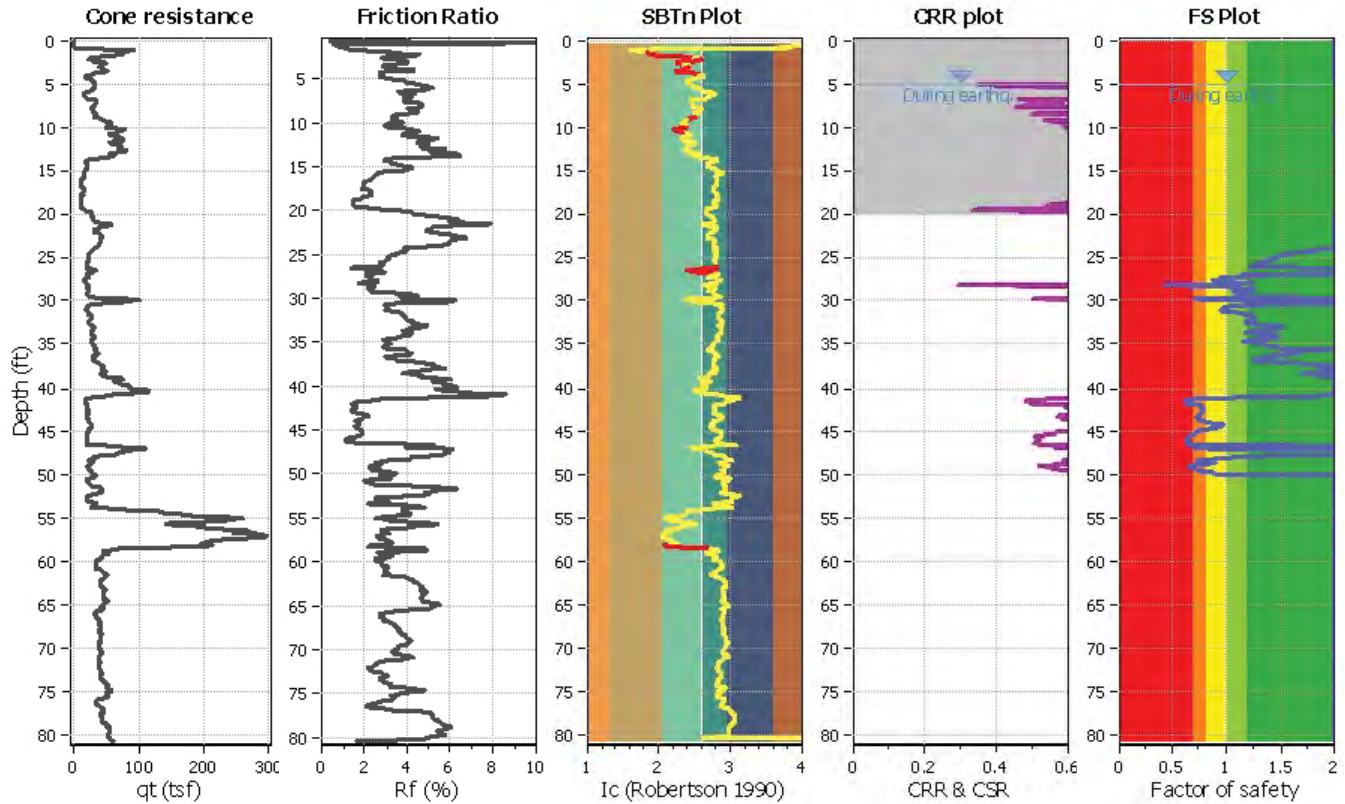
Project title : 1095 Rollins Road, 14140.000.000

Location : Burlingame, California

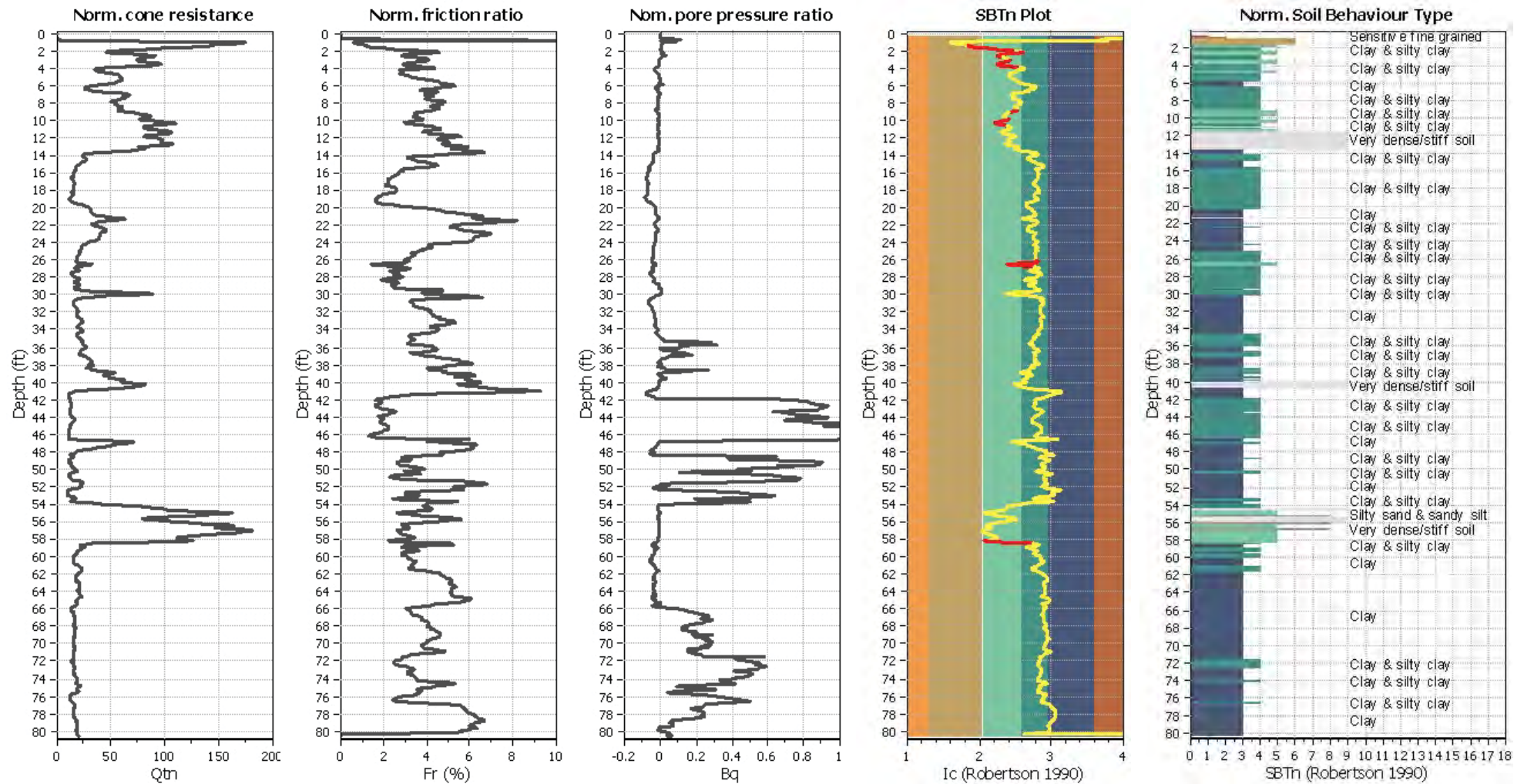
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Excavation:	Yes	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Excavation depth:	20.00 ft	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	2.00 tsf	Limit depth applied:	Yes
Earthquake magnitude M_w :	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_0 applied:	No	MSF method:	Method based



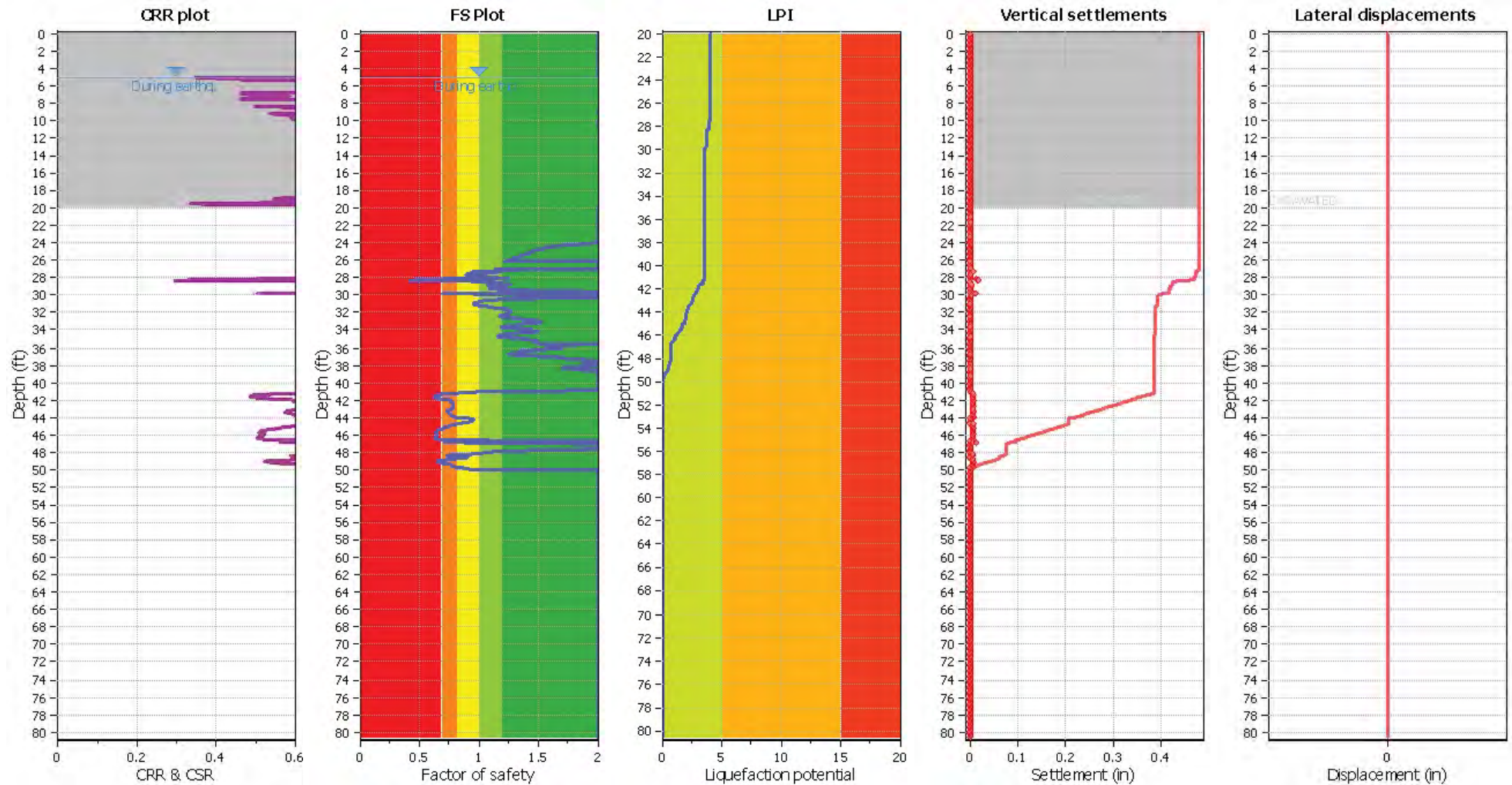
CPT basic interpretation plots (normaliz



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	5.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.82	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Excavation depth:	20.00 ft	Limit depth:	50.00 ft

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (earthq.):	5.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I _c value	I _c cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.82	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Excavation depth:	20.00 ft	Limit depth:	50.00 ft

F.S. color scheme

Red	Almost certain it will liquefy
Orange	Very likely to liquefy
Yellow	Liquefaction and no liq. are equally likely
Light Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

LPI color scheme

Red	Very high risk
Orange	High risk
Yellow	Low risk

LIQUEFACTION ANALYSIS REPORT

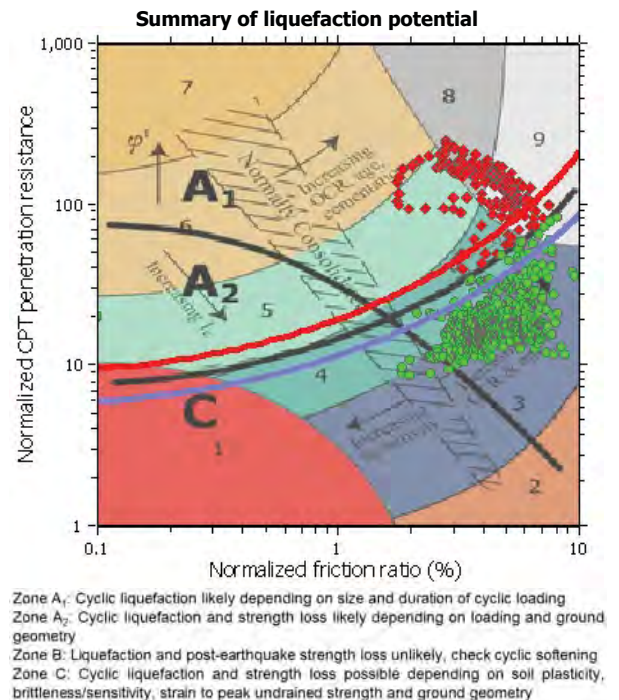
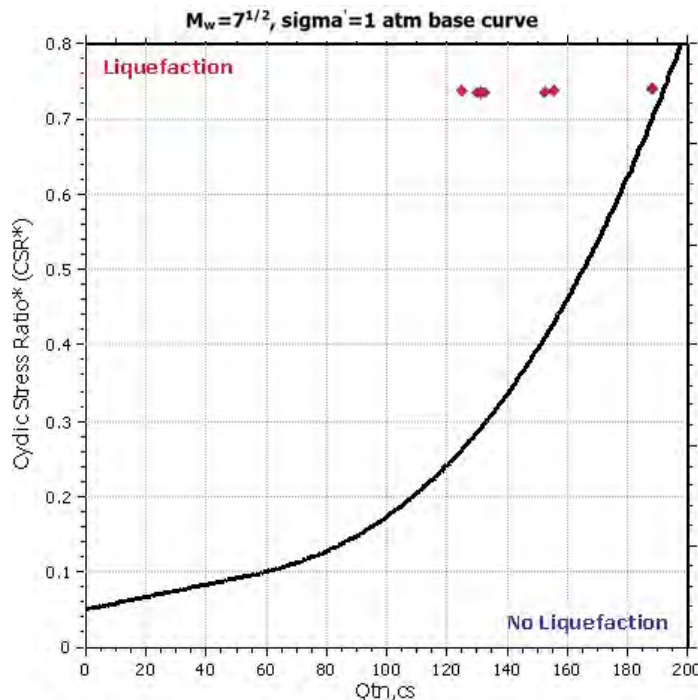
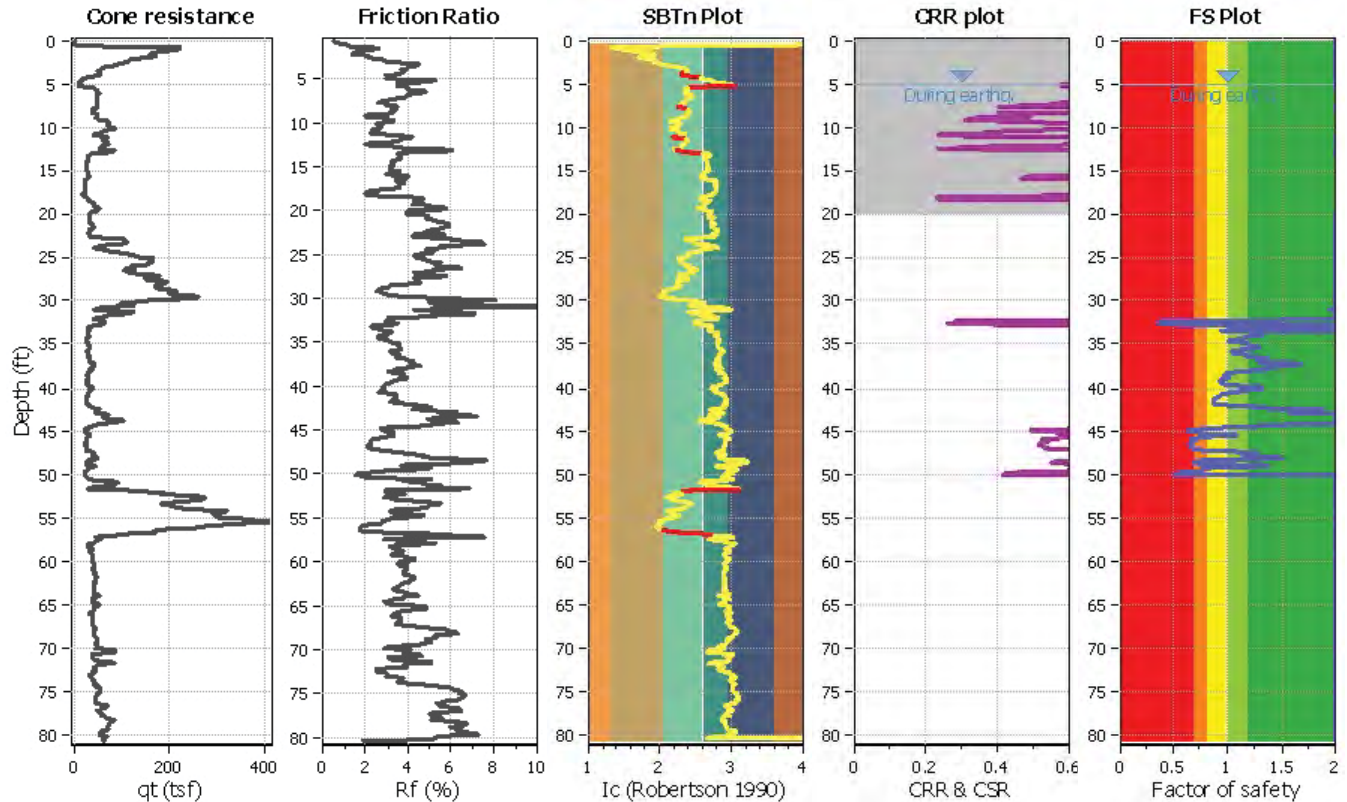
Project title : 1095 Rollins Road, 14140.000.000

Location : Burlingame, California

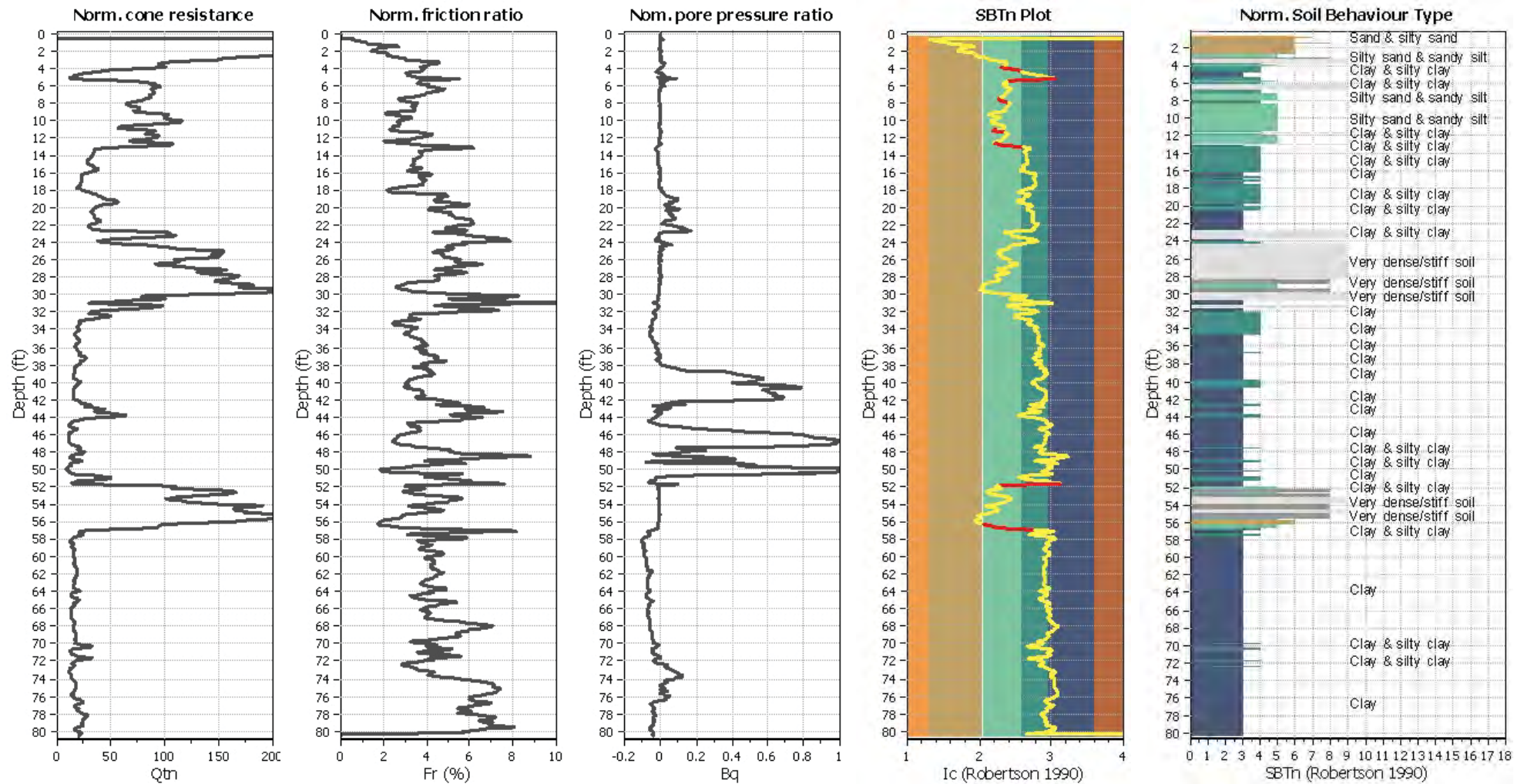
CPT file : 1-CPT3

Input parameters and analysis data

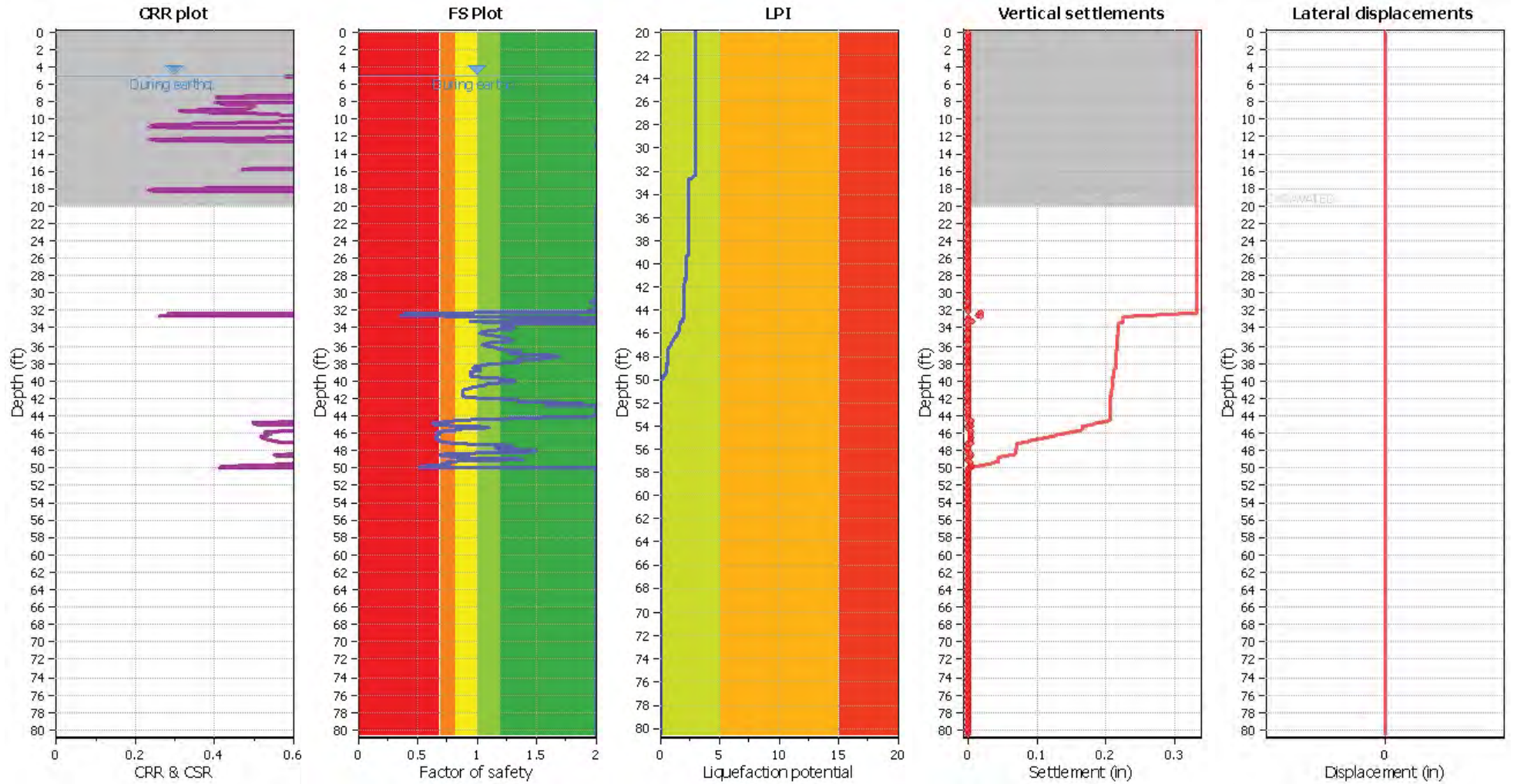
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Excavation:	Yes	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	5.00 ft	Excavation depth:	20.00 ft	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Footing load:	2.00 tsf	Limit depth applied:	Yes
Earthquake magnitude M_w :	8.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.82	Unit weight calculation:	Based on SBT	K_0 applied:	No	MSF method:	Method based



CPT basic interpretation plots (normaliz



Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (earthq.):	5.00 ft	Footing load:	2.00 tsf
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	No
Earthquake magnitude M _w :	8.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.82	Excavation:	Yes	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Excavation depth:	20.00 ft	Limit depth:	50.00 ft

F.S. color scheme

Red	Almost certain it will liquefy
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Yellow	Liquefaction and no liq. are equally likely
Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

LPI color scheme

Red	Very high risk
Orange	High risk
Yellow	Low risk



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